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Expansion of District of Columbia Water System

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BEFORE the impact of the war had increased the population of the national capital area by some 320,000 persons, to make a total of 1,041,000, studies for an increase in water supply had been initiated.

In a way, it was fortunate that the 44 per cent population increase—one of the greatest increases in any city in the country—occurred while the studies, planning and programming were in progress. It permitted a perspective view of future water needs that otherwise could not have been envisioned, as, in Washington and its metropolitan area, a large part of such influxes of peoples usually remain as permanent residents.

Today, the maximum demand for filtered water exceeds the rated or nominal capacity of the two filtration plants, and the capacity of the two major pumping plants has practically been reached.

This paper explains what has been done to take care of the tremendous increase in population and why Congress issued a directive for the preparation of a report "for the development of a plan to insure an adequate future water supply for the District of Columbia. . . ."

A little background information will probably aid the understanding of the improvement program discussed herein.

One phase of the operation of the Washington water system is unique: for the supply system, called the Washington Aqueduct, is under the control of the War Dept. and operated by the U.S. Engineer Office, Washington, D.C.; and the distribution system is

under the control of the Board of Commissioners of the District of Columbia and is operated by the Water Div. of the municipal government.

Water revenues are collected by the city and placed in a special Water Fund in the U.S. Treasury. The Congress appropriates annually from this fund the sums required by both the supply and distribution divisions of the water system for operation and maintenance and the improvements which constitute capital expenditures. There is complete co-ordination between the two divisions in the functioning of the water system.

Federal control of the supply division of the water system goes back almost 100 years to the time when there was no water system, and the population was supplied by springs and wells. At that time, Congress directed that a report be prepared on a water supply primarily for the federal establishments and, to the extent feasible, for the citizens of Georgetown and Washington. This report was prepared by the Army Engineers, who also began, in the 1850's, the construction of the first water works. This system included a diversion dam in the Potomac River at Great Falls, Md.; 11 miles of 9-ft. diameter circular brick conduit; a receiving reservoir; a distributing reservoir permitting natural settling; and certain distribution mains and pipes within the city proper. Except for three years during the Civil War, the Army Engineers have continued to be responsible for the National Capital's water supply. It was not until 1882 that the city authorities began actively to distribute water to Washington residents.

In 1853, it was believed that the Washington Aqueduct system, as then planned, would be adequate for all time. By 1902, however, turbid water could no longer be tolerated, and typhoid and other health hazards became too great a menace. In 1905, the 75-mgd. slow sand plant at McMillan, located in the center of the city, was placed in service and the large Bryant Street high-lift pumping station was built.

Expectations of a long-life capacity of supply and filtration failed to materialize, due largely to the population increase of World War I. During the 1920's, therefore, a new 80-mgd. rapid sand filter plant was built at Dalecarlia on the westerly boundary of the District of Columbia, together with a new 9-mile raw water conduit, a pumping station, transmission mains, and high-service reservoirs. The distribution system, correspondingly, was extended and reinforced.

The Dalecarlia expansion was to have lasted until 1980, but the influx of people to the National Capital during World War II, as noted previously, again upset all calculations by about 35 years. Already, a beginning has been made on the third major expansion of the water system of the District of Columbia. Figure 1 shows the existing water system and the proposed improvements in diagrammatic form.

It may be well to explain at the outset that the District of Columbia water system supplies the whole of Arlington County, Va., with filtered water and also certain adjacent Maryland areas with a very limited quantity of water. Arlington County is that part of the original District of Columbia, as planned by George Washington and Pierre L'Enfant, which was retroceded to Virginia in 1846. The area embraced by the entire water system, including Arlington County, extends

20 miles east-to-west and 13 miles north-to-south. Recently, a bill was introduced in Congress to furnish water from the District of Columbia water system to a part of Fairfax County, Va., which is located beyond Arlington, within the Washington metropolitan area.

Pursuant to the directive from Congress, the District Engineer of the U.S. Engineer Office and the Engineer Commissioner of the District of Columbia, with their respective water works staffs, prepared a joint report which develops and describes a comprehensive plan and program for the construction of improvements and additions to the water system of the District of Columbia and the metropolitan area, including Arlington County. The plan provides for the necessary facilities to produce and to distribute a potable water to meet safely and adequately the anticipated future demands due to population growth during the next half century. It contemplates an over-all expenditure of about $22,000,000 for supply facilities and about $19,000,000 for distribution facilities during the ensuing 43 years. The principal works, however, will be constructed during the next 14 years.

Population

Planning to increase the facilities of the water system of the District of Columbia basically involves a study of population growth in order to anticipate with reasonable accuracy the water consumption at some future date.

It is estimated that population saturation within the District of Columbia and Arlington County, under existing zoning ordinances, will occur about the end of the present century; hence, the year 2000 has been selected as the future date for which to plan. Extensive studies were made of the population trends of the District of Columbia and Arlington County, as these areas are supplied wholly with water from the District of Columbia water system. General consideration was given to Alexandria, Va., and to those communities of Fairfax County which are adjacent to Arlington County and which, in the future, might become users of water supplied by the District of Columbia water system.

In preparing estimates of the future population of Washington, D.C., an extensive study was made to compare the city with other cities of the United States, but it was found that Washington has developed characteristics, not common to any other city, that reflected the growth of the Federal Government in national and international affairs, and the growing tendency of business, trade and other national and foreign organizations to locate their headquarters in the city. For these reasons, it was concluded that the future population growth of the city would be influenced by its peculiar character and would not follow the pattern of growth of other cities.

The past development of Washington has shown that wars and other conditions affecting the national economy produce a decidedly upward inclination in the population curve. This growth is to be expected because Washington is the seat of the national government, and expansion in federal activities correspondingly affects the growth of the District of Columbia and its immediate environs. For example, during the depression of the 1930's, the District of Columbia experienced a material growth in population. During World War II, great expansion in federal activities and employment brought about a percentage of increase

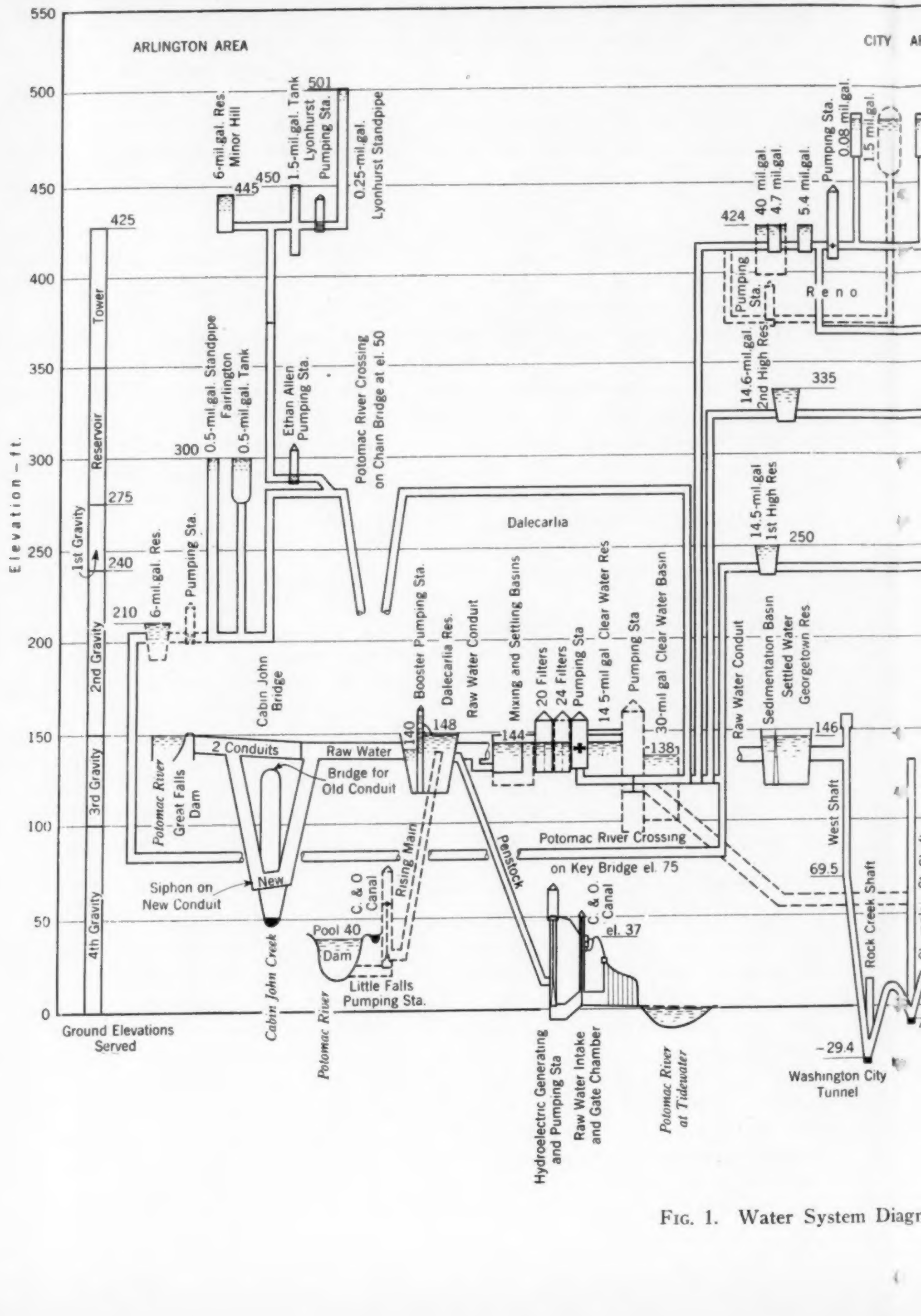
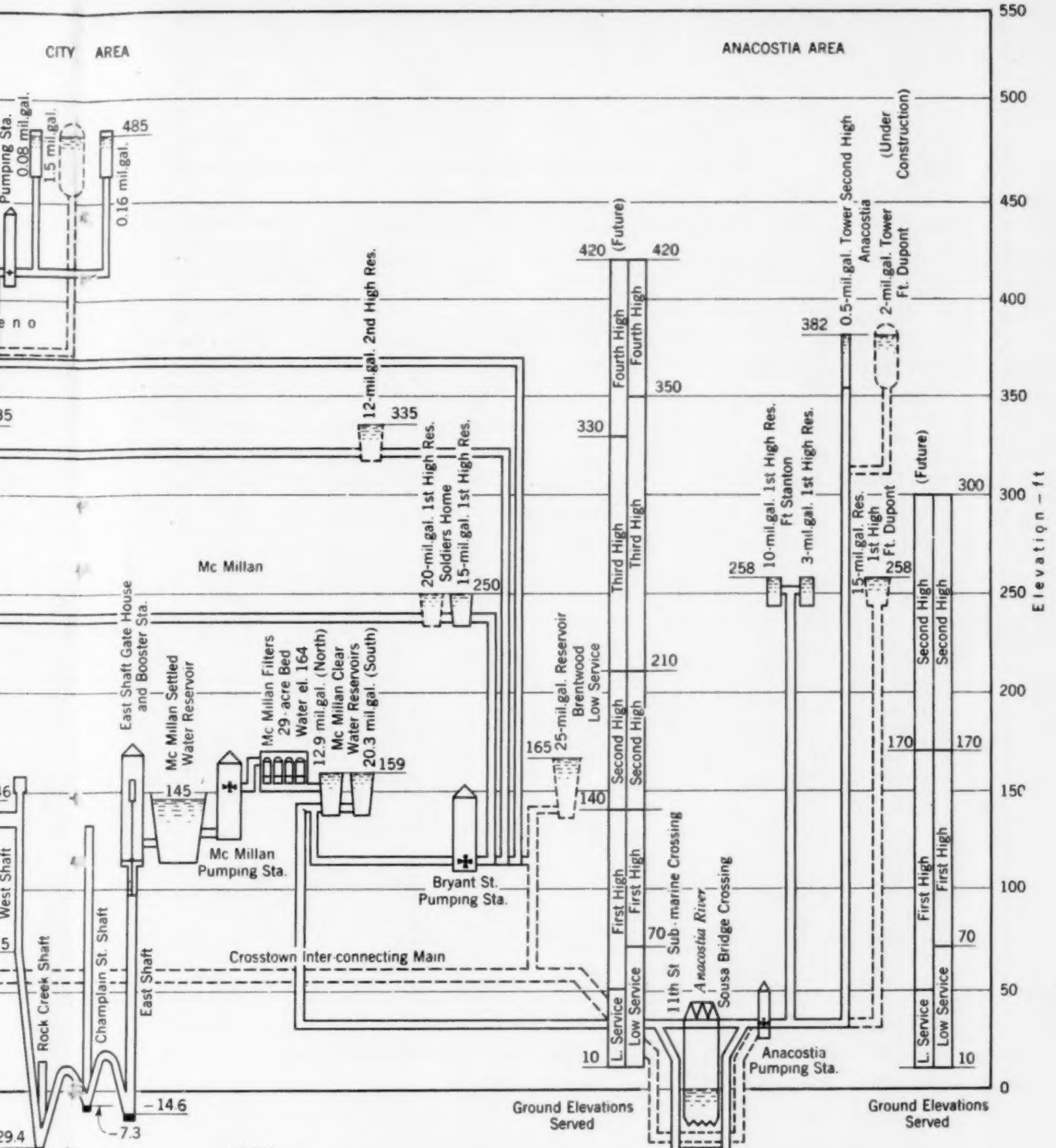


FIG. 1. Water System Diagr



NOTES:

Elevations of Water in Various Structures Shown at Normal and Maximum High Elevations.

Existing Facilities Shown in Solid Lines.

Future Improvements Shown in Dotted Lines.

Elevations for Washington Aqueduct Structures Based on W. A. Datum = 0.94 Ft. Above Mean Sea Level.

Elevations for Water Dept. Structures Based on Dist. of Columbia Datum = 0.70 Ft. Above Mean Sea Level.

stem Diagram—Present and Future

in population in Washington not equaled by any other city of comparable size in the United States. This wartime increase will greatly affect the future rate of growth of the city and its metropolitan area.

After a careful examination of numerous standard methods of predicting future population growth, a procedure was selected which is predicated upon the special characteristics influencing local growth, the extent of habitable areas, the present densities of occupied areas, the determination of unoccupied habitable areas and the assignment of reasonable future densities.

As the District of Columbia is a Federal territory, its boundaries differ from those of most cities in being fixed by law and in not being capable of change except by an amendment to the Constitution. These fixed boundaries permit a much better population forecast than would be possible if annexation of adjacent territory were readily effected. Taking into consideration the existing zoning regulations and making some allowances for changes, possible increased federal holdings in the city and the periodical occurrence of national crises, it was possible to build up a saturation population. Another guide for population estimates in Washington is federal employment and the changes in the ratio of the supporting population to the federal worker. This ratio (Fig. 2) was 2.9 at the peak of employment during World War II and about 6.0 during the prior peacetime period. The determination of the time when this saturation population would be reached presented quite a problem.

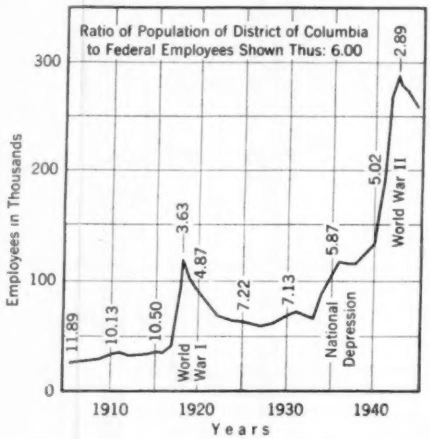


FIG. 2. Growth of Federal Employment

Practically all cities which have reached maturity have followed an **S**-curve line of growth because, as saturation is approached, the smaller areas remaining unoccupied are slower in developing, and the curve flattens out. The past and estimated future population of the District of Columbia and Arlington, Va., is shown in Fig. 3.

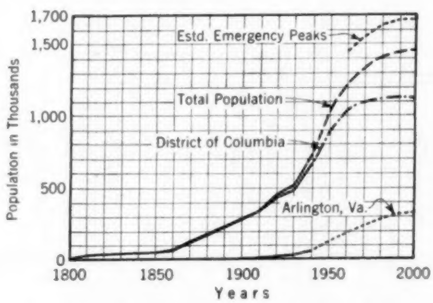


FIG. 3. Population Trends—District of Columbia and Arlington, Va.

The District of Columbia, as a federal city with very few industrial establishments, has a pattern of population growth not comparable with other capital cities. Although there may be some factors, such as zoning ordinance changes permitting vertical growth, which may advance or postpone the time of maturity, it appears that sat-

uration in the District of Columbia will occur at the end of the present century. The saturation population at the year 2000 is estimated at 1,125,000 for the District of Columbia and 330,-000 for Arlington, a total of 1,455,000. This represents normal occupation of land areas without crowding. Should a national emergency such as World War II occur after saturation is reached, however, the normal future population of 1,455,000 would increase, probably to 1,680,000.

Water Consumption

Concomitant with the estimate of population growth of the Washington area, extensive studies of the water use characteristics were conducted, as these are of the utmost importance in fixing the capacities and sizes of all parts of the water system. As a basis for these studies, the estimated future increase in population of the District of Columbia was apportioned to the water service areas according to the land available for population expansion. Population densities were based upon an analysis of present densities in improved areas, existing zoning ordinances and special service-area characteristics which would influence future growth.

TABLE 1

Ratios of Maximum Day Consumption to Annual Average Day Consumption

Year	Service Areas						Washington, D.C.	Arlington, Va.	Washington, D.C., and Arlington, Va.
	Low	First High	Second High	Third High	Fourth High	Anacostia First and Second High			
1930	1.22	1.32	1.42	1.96	1.36	1.22	1.27	2.51	1.28
1931	1.26	1.71	1.30	1.73	1.87	1.61	1.27	2.19	1.28
1932	1.25	1.25	1.41	1.78	2.46	1.74	1.37	2.07	1.38
1933	1.22	1.23	1.25	1.57	1.62	1.59	1.25	2.17	1.25
1934	1.24	1.52	1.43	1.82	1.67	1.81	1.34	3.77	1.37
1935	1.27	1.36	1.40	1.66	1.53	1.62	1.36	1.83	1.36
1936	1.31	1.33	1.41	2.03	1.91	1.38	1.42	2.03	1.44
1937	1.39	1.25	1.32	1.58	2.05	1.58	1.31	1.71	1.31
1938	1.34	1.24	1.30	1.68	1.96	1.40	1.38	1.44	1.38
1939	1.32	1.24	1.45	1.77	2.24	1.75	1.36	1.80	1.38
1940	1.39	1.34	1.42	1.65	2.25	1.62	1.43	1.66	1.43
1941	1.32	1.30	1.41	1.54	2.15	1.47	1.32	1.51	1.32
1942	1.31	1.31	1.26	1.58	2.18	1.78	1.33	1.25	1.33
1943	1.45	1.39	1.39	1.53	2.02	1.56	1.36	1.44	1.37
1944	1.39	1.28	1.37	1.36	2.27	1.47	1.36	1.36	1.36
1945	1.29	1.35	1.31	1.43	1.56	1.48	1.24	1.36	1.23
Max.	1.45	1.71	1.45	2.03	2.46	1.81	1.43	3.77	1.44
Avg.	1.31	1.34	1.37	1.67	1.94	1.57	1.33	1.88	1.34
Min.	1.22	1.23	1.25	1.36	1.36	1.22	1.24	1.25	1.23

Consumption of water in the District of Columbia and its environs falls into four major categories: (1) domestic or private use, (2) commercial use, (3) federal government use and

(4) municipal government use. Unlike other cities, Washington requires a large quantity of water to supply the vast federal establishments and their worker populations located in the District of Columbia and the adjacent areas of the states of Maryland and Virginia. The municipal use of water in the District of Columbia is quite substantial. Of the average daily consumption of 150 mil.gal. for all consumers in 1945, the federal government used 25.5 mil.gal. and the District of Columbia government used 6.7 mil.gal.

Per capita water consumption within the District of Columbia and Arlington County varies considerably, not only over the whole area, but also between the several water service areas, according to their particular characteristics. Some of these characteristics are:

1. Class of inhabitants
2. Type of dwellings
3. Size of lawns and gardens
4. Extent of federal and municipal activities
5. Prevalence of out-of-town water users, including those on commercial and governmental business and tourists
6. Type and extent of commercial activities
7. Adequacy and pressure of distribution system
8. Extent of sewage system
9. Extent of water-using facilities
10. Climatological conditions.

The annual average daily consumption in Washington, although it is a

TABLE 2

Population and Water Consumption

Year	Population		Water Consumption *mgd.*				Per Capita Consumption *gpd.*			
			Ann. Avg. Day		Max. Day		Ann. Avg. Day		Max. Day	
	Dist. of Columbia	Arlington	Dist. of Columbia	Arlington	Dist. of Columbia	Arlington	Dist. of Columbia	Arlington	Dist. of Columbia	Arlington
1930	487,000	26,000	85.6	0.6	109.0	1.5	176	23	224	58
1931	504,000	29,000	86.7	0.6	110.2	1.5	172	21	219	52
1932	513,000	32,000	86.9	0.8	119.2	1.6	169	25	232	50
1933	529,000	35,000	85.6	0.8	106.7	1.7	162	23	202	49
1934	568,000	38,000	91.7	1.0	123.1	3.8	161	26	217	100
1935	608,000	41,000	94.2	1.3	127.7	2.4	155	32	210	59
1936	629,000	44,000	101.1	1.8	144.4	3.6	161	41	229	82
1937	616,000	47,000	98.5	2.0	128.8	3.4	160	43	209	72
1938	638,000	50,000	99.6	2.4	137.2	3.5	156	48	215	70
1939	658,000	53,000	101.9	2.8	139.1	5.1	155	53	211	96
1940	663,000	57,000	103.0	3.6	147.4	6.0	155	63	222	105
1941	746,000	72,000	115.5	4.5	152.8	6.8	155	63	205	94
1942	809,000	83,000	120.3	5.0	160.3	6.3	149	60	198	76
1943	866,000	105,000	131.7	6.2	179.5	8.9	152	59	207	85
1944	896,000	125,000	139.8	7.1	190.5	9.7	156	57	213	78
1945	906,000	135,000	142.3	7.1	175.8	9.7	157	53	194	72

theoretical quantity which may be valid only by coincidence, is used as a yardstick for predicting maximum consumptions by the use of ratios based upon past experience. Table 1 shows the variations in the ratios of the maximum day to the annual average day within the several service areas under actual conditions for the years 1930 to 1945, inclusive.

In order to assure a reasonably accurate forecast of the future water demand, analyses were made of the records of water consumption for Washington and Arlington in relation to the ratios given in Table 1 and the per capita rates of consumption. The two census years with accurate population enumerations, together with measured quantities of water, served as a check of the intercensal estimates. Table 2 shows the data for the years 1930 to 1945, inclusive.

A meter installation program was begun in 1904 and was advanced so actively that by 1917 the city was 85 per cent metered. In 1931, another metering program was initiated, and, from the middle 1930's to the advent of World War II, the total was increased to about 94 per cent. Under these circumstances, the basic consumption data for forecasting the water demand were accurate.

The average per capita consumption in the District of Columbia for 1930 was substantial, but an unusually hot and dry summer influenced the results. On the other hand, the latter years of the 1930–1945 period show comparatively lower per capita rates, reflecting the overcrowding caused by the large increase in federal employment. During these years, the rate of population increase proceeded at a greater pace than the increase in water utilities—in other words, the individual opportunity to use water was somewhat restricted. Table 2 also shows low per capita rates for Arlington, particularly in the early years of the period, when that community was using water partly from the water system of the District of Columbia and partly from private wells which since have been abandoned. The county is now rapidly approaching the status of a city, however, and is on a rising scale of per capita consumption. This condition was given weight in estimating future water quantities.

An important feature affecting the prediction of the future demand for water within the water service areas is the plan to alter or to modify the boundaries of the areas as future development takes place, in order to obtain optimum pressures and flow conditions within the distribution system. These changes are to take place, as the occasion dictates, prior to 1970.

The estimated population of the District of Columbia and Arlington for the year 2000, together with the anticipated per capita rates of water consumption, determines the future demand which the water supply system will be called upon to meet. Table 3 shows the actual population and water consumption for the census years 1930 and 1940 and the estimated future population and water consumption by decades through the year 2000. The changes in the boundaries of the water service areas, to be made between 1950 and 1970, are taken into account in the tabulation.

Table 3 indicates the anticipated water consumption based upon normal population growth and average operating conditions; however, past experience indicates that there will be occasions when abnormal conditions will prevail, either to increase normal wa-

TABLE 3

Past and Estimated Population and Water Consump-

Water Service Area	1930			1940			1950			1960		
	Pop.	Water Consump. *mgd.*		Pop.	Water Consump. *mgd.*		Pop.	Water Consump. *mgd.*		Pop.	Water Consump. *mgd.*	
		Ann. Avg. Day	Max. Day		Ann. Avg. Day	Max. Day		Ann. Avg. Day	Max. Day		Ann. Avg. Day	Max. Day
Low service	135,874	33.90	41.80	170,484	40.60	56.60	191,700	47.00	63.30	192,900	43.60	60.40
First high	135,208	21.50	27.80	171,565	27.40	36.70	184,000	29.40	38.70	271,200	44.70	60.70
Second high	119,453	17.20	23.60	163,173	19.50	28.70	190,600	25.10	35.40	202,300	27.50	38.90
Third high	65,439	9.90	18.90	98,402	13.50	23.60	160,000	23.20	39.40	167,400	24.10	41.00
Fourth high	9,211	1.65	3.25	17,404	2.20	4.95	26,200	4.20	7.90	28,200	4.50	9.00
Anacostia												
First high	13,386	0.89	1.10	26,249	2.12	3.44	85,500	8.60	13.80	104,300	12.30	19.80
Second high	8,298	0.56	0.68	15,814	1.28	2.08	55,000	5.50	8.80	65,700	7.70	12.50
TOTAL												
Washington, D. C.	486,869	85.60	109.00	663,091	106.60	147.40	893,000	143.00	200.10	1,032,000	164.40	232.30
Combined max.*			117.13			156.07			207.30			242.30
Arlington, Va.	26,069	0.57	1.13	57,040	3.57	5.91	125,000	11.90	21.40	188,000	22.00	38.00
GRAND TOTAL	512,938	86.17	110.13	720,131	110.17	152.60	1,018,000	154.90	221.50	1,220,000	186.40	267.20
Combined max.*			118.26			161.98			228.70			280.30

* Maximum-day water consumptions which will occur if the maxima of all service areas

ter consumption or decrease the water output; or both of these conditions may occur simultaneously. Some of these contingencies are:

1. National emergency, either economic or military, which will cause a sudden increase in population.
2. Periods of drought and high temperature, causing sharp increases in the water demand.
3. Wide private and commercial use of air-cooling and refrigerating equipment.
4. River floods and unusual outbreaks of micro-organisms.
5. Temporary reductions in output due to accident, such as physical breakdowns and power failures.

Such contingencies are not susceptible to specific evaluation in terms of water consumption; however, they may impose an additional demand of 25 per cent, or 85 mgd. (Fig. 4). To meet the maximum daily demand by the year 2000, the water supply system must have a nominal capacity of 342 mgd. Such a system could be overloaded to supply a total of 427 mgd. for limited periods without impairment, and this overload capacity will provide for contingent increases in demand over the estimated maximum.

The increase in water consumption to the year 2000 and the increments in capacity to be built into the raw and filtered water supply systems to meet the demands in the future are shown graphically in Fig. 4.

Collecting Facilities

Raw water is obtained from the Potomac River at Great Falls, Md., above which the watershed comprises an area

TABLE 3

tion of Washington, D.C., and Arlington County, Va.

1970			1980			1990			2000		
Pop.	Water Consump. *mgd.*		Pop.	Water Consump. *mgd.*		Pop.	Water Consump. *mgd.*		Pop.	Water Consump. *mgd.*	
	Ann. Avg. Day	Max. Day		Ann. Avg. Day	Max. Day		Ann. Avg. Day	Max. Day		Ann. Avg. Day	Max. Day
178,300	37.60	51.30	182,300	38.40	52.30	184,100	38.80	52.90	184,100	38.80	52.90
336,200	57.20	80.20	343,300	58.30	82.00	344,800	58.70	82.50	344,800	58.70	82.50
198,000	26.90	38.20	202,500	28.50	40.50	205,400	28.90	41.00	205,400	28.90	41.00
161,400	23.10	39.30	165,600	23.70	40.20	167,600	24.10	40.70	167,600	24.10	40.70
28,800	4.60	9.20	29,400	4.70	9.40	29,800	4.80	9.50	29,800	4.80	9.50
114,900	15.50	24.80	117,500	15.90	25.40	118,600	16.00	25.60	118,600	16.00	25.60
72,400	9.80	15.60	73,400	9.90	15.80	74,700	10.10	16.10	74,700	10.10	16.10
1,090,000	174.70	245.00	1,114,000	179.40	250.80	1,125,000	181.40	253.00	1,125,000	181.40	253.00
		258.60			265.60			268.30			268.30
240,000	31.00	52.50	286,000	37.70	64.40	317,000	41.80	71.30	330,000	43.60	74.30
1,330,000	205.70	292.00	1,400,000	217.10	306.60	1,442,000	223.20	316.00	1,455,000	225.00	319.00
		311.10			330.00			339.60			342.60

fall on the same day.

of 11,400 square miles. The average discharge of the river at the intakes is about 7,500 mgd., with a recorded minimum discharge of 506 mgd. for one day only. The maximum daily demand to the year 2000 is estimated at 342.6 mgd., with a possible contingent increase to 427 mgd.; hence, the river discharge is adequate even at the recorded minimum one-day flow.

Existing raw water collecting facilities consist of a substantial masonry diversion dam at el. 150.5 mean sea level across the Potomac River at Great Falls. A pool is created from which water is taken through two sluice-gate controlled intake chambers, fed into two masonry conduits, and conveyed by gravity over a distance of nine miles to the forebay of the Dalecarlia receiving reservoir. From the Dalecarlia forebay at el. 141.0, booster pumping equipment located in a structure forming a part of the dam across the northern neck of the reservoir lifts the water through a vertical distance of 7 ft. to the main body of the Dalecarlia reservoir at el. 148.0, from which raw water is supplied to the Dalecarlia and McMillan filter plants. In addition, there is the hydroelectric conversion pumping plant obtained as a wartime security measure through converting one of the two hydroelectric generating station turbines to a pumping unit of fair efficiency by rewiring the generator for use as a motor and installing a pump impeller in the turbine casing. An intake was provided to obtain water from the Chesapeake and Ohio Canal at el. 39.0. Equipment is on hand to convert the second turbine should an emergency require it.

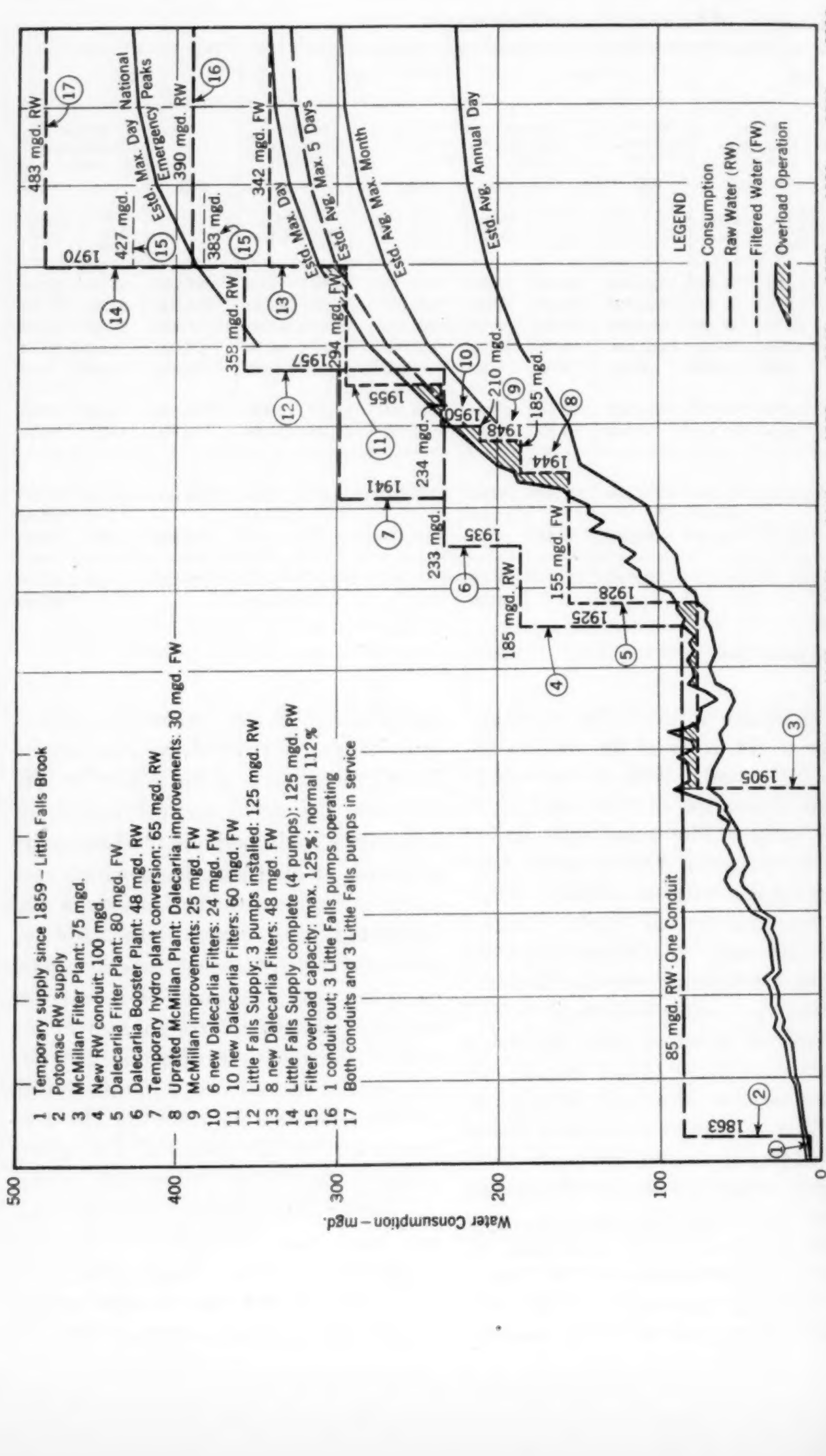


FIG. 4. Water Consumption and Supply

With the booster pumping plant in operation, the maximum quantity of raw water which the conduits from Great Falls will deliver is 233 mgd. The hydroelectric conversion pumping station will deliver raw water at the rate of 65 mgd. with one converted unit in operation, or 120 mgd. if both units are made to operate. As this is a temporary plant, however, and the Chesapeake and Ohio Canal is not a fully dependable source of raw water, it cannot be considered a part of the permanent supply. The only existing permanent means of transmitting raw water is the two conduits, which have a maximum capacity of 233 mgd. This capacity is dependable only if the booster pumping plant is working to capacity, for an interruption in operation will flatten the gradient through the conduits, causing a corresponding reduction in flow.

Experience has shown that the factor most likely to cause interruption in service is the repair of a section of the new conduit, leaving an available quantity of 140 mgd. Such repairs have been necessary a number of times, when settlement and slides caused leaks. On two occasions, a section of the new conduit was out of service for a period of 30 days. During these periods, it was necessary to pass the entire water supply through the old conduit for about one-third of its length; the resulting reduction in flow amounted to about 93 mgd. If a failure makes it necessary to take the entire new conduit out of service, however, the reduction in flow would be 127 mgd., and the resulting supply of raw water would be 106 mgd.

The quantity of raw water available, exclusive of the emergency use of the hydroelectric conversion pumping units, if operational or physical contingencies should cause a reduction of flow, is shown in Table 4.

Improvements to the purification system will result in a total nominal capacity of 342 mgd. for the two filter plants, with an overload capacity of 427 mgd. The quantity of raw water available with one section of the new conduit out of service is 140 mgd., as shown in Table 4, and the deficiency in the required 427-mgd. raw water supply will be 287 mgd. Therefore,

TABLE 4

Raw Water Available Through Conduits Under Contingent Conditions

Contingency	Reduction in Flow *mgd.*	Quantity Available *mgd.*
Shut-down of Dalecarlia booster plant	48	185
Section of new conduit out of service	93	140
Entire new conduit out of service	127	106
Both conduits out of service	233	0

additional raw water collecting facilities of about 290-mgd. capacity are to be provided to develop the full overload capacity of the filter plants for use when the present supply is interrupted.

Several schemes for obtaining an additional raw water supply were analyzed, and it was found that the most economical procedure would be to pump from the nearby Potomac River to the Dalecarlia reservoir.

About 1¼ miles northwest of the Dalecarlia filter plant, at the head of Little Falls on the Potomac River,

there is a loose rock-fill feeder dam which diverts water to the Chesapeake and Ohio Canal. This dam will be replaced with a permanent concrete structure to form a pool at el. 40.0 for supplying raw water to a new pumping station located on the east bank of the river. From this pool, water will be pumped through a rising tunnel about 10 ft. in diameter to the Dalecarlia reservoir. The pumping station is to have an installed capacity of 425 mgd., inclusive of stand-by units.

Purification Facilities

As the Potomac River, the raw water source, is turbid and polluted, adequate purification facilities are of prime importance. The Potomac is a flashy river with turbidity ranging from a minimum of 4 to a maximum of 5,200 ppm. Comparable micro-organism counts range from a minimum of 40 to 77,000 per ml.

There are two existing filter plants in the supply system: the McMillan slow sand filter plant, which began service in 1905, and the Dalecarlia rapid sand filter plant, which began service in 1928.

The McMillan plant consists of 29 filters with a net sand area of one acre each. The original nominal capacity of the plant was 75 mgd. Because of the great population increase in the city during World War II, however, it became necessary to exceed the nominal plant capacity. By 1945, improvements in the hydraulic grade of the system, the introduction of mechanical sand washing equipment and the continuous use of aluminum sulfate to obtain better clarification of the applied water had increased the nominal capacity of the plant from 75 to 100 mgd., and the overload capacity to 110 mgd.

The Dalecarlia filter plant consists of 20 rapid sand filters with an originally rated total capacity of 80 mgd. Since its construction, minor improvements have been made, especially in the installation of new under-drain systems and surface wash equipment, increasing the plant output somewhat. The ability of the plant to operate at overload rates, however, is limited to the head differential which must be maintained between the levels of the water on the filters and in the clear water basin. The present nominal capacity of the plant is 85 mgd., with an overload capacity, which can be sustained for a short period, of 95 mgd.

The anticipated future demand of the District of Columbia and Arlington, to the year 2000, has been shown to be 342 mgd. for the maximum day with normal population expansion plus a 25 per cent additional demand to meet abnormal conditions, or a total over-all maximum demand of 427 mgd. The present purification facilities have a nominal capacity of 185 mgd. and, hence, additional capacity of 157 mgd. will be provided to meet the future demand of 342 mgd., with reliance upon overload capacity to meet the maximum anticipated contingent demand of 427 mgd.

The determination of the proper site for the improvements and additions to the purification facilities of the water supply system involved the economic consideration of capital costs of construction, carrying charges and operating costs, and, for this purpose, four separate schemes, designated A, B, C and D, were conceived and analyzed.

Scheme A, in general, contemplated enlargement of the present Dalecarlia plant to increase its nominal capacity from 85 to 217 mgd. and certain modifications and improvements in the Mc-

Millan plant to increase its nominal capacity from 100 to 125 mgd.

Schemes B, C and D planned the major increases in plant capacity at the McMillan filter plant. The nominal capacity was to be increased from 100 to 233 mgd. by the construction of a rapid sand plant or a modified with the construction of six additional filters and appurtenances as provided for in the original plans.

Economic comparison of the several schemes was made on the basis of the costs of construction; electric power, chemical treatment, administrative overhead, labor and other operat-

TABLE 5

Economic Comparison of Schemes for Improvements and Additions

	Scheme A	Scheme B	Scheme C	Scheme D
Capital cost	$16,222,000	$20,455,000	$19,439,000	$19,154,000
Increased Annual Operating and Nonoperating Costs, 2000 Over 1942:*				
Operating costs				
Labor	$ 82,000	$ 143,900	$ 160,600	$ 128,200
Electric power	249,600	249,600	249,600	249,600
Chemicals	95,800	95,800	95,800	95,800
Administrative overhead	37,000	37,000	37,000	37,000
All other operating costs	64,900	70,000	58,000	67,000
Total operating costs	$ 529,300	$ 596,300	$ 601,000	$ 577,600
Nonoperating costs				
Interest on capital cost†	$ 486,700	$ 613,700	$ 583,200	$ 574,600
Annual depreciation accrual‡	117,200	140,300	120,900	130,600
Total nonoperating costs	$ 603,900	$ 754,000	$ 704,100	$ 705,200
Grand total additional annual costs	$1,133,200	$1,350,300	$1,305,100	$1,282,800
Total extra operating and nonoperating cost over Scheme A during composite life of facilities	None	11,940,500	9,454,500	8,228,000

* The year 1942 was selected as representative of the best average condition in recent years.

† Interest computed at 3 per cent per annum on capital cost.

‡ Annual depreciation accrual based upon the sinking-fund method with interest accretion at 3 per cent per annum.

rapid sand plant under schemes B and D, or reconstruction of the slow sand filter plant under scheme C. Each of these schemes involved a new tunnel 5 miles long to bring the additional raw water to McMillan. Any scheme, however, would increase the existing Dalecarlia plant capacity by 24 mgd.

ing costs; interest; and depreciation. The operating and nonoperating costs developed are the additional costs of the year 2000 over the year 1942. The year 1942 was taken as representative of normal conditions, and actual costs were taken from existing plant records. Only the additional costs are

considered pertinent in the economic comparison of the several schemes, as the costs for the year 1942 are common to all. Furthermore, all items of capital improvement common to all of the schemes were eliminated. The economic comparison of the several schemes is given in Table 5.

It will be observed that the improvements to the water supply system under scheme A are lower both in first cost and in annual operating and non-operating costs; hence the Dalecarlia site was selected as the place where future major improvements and additions to the purification system will be made. In addition to the enlarged filtration and sedimentation facilities, a chemical building containing expanded equipment for the manufacture, storage and application of chemicals and laboratory facilities will be constructed. The adopted improvements and their estimated costs are presented in Table 6.

TABLE 6

Proposed Improvements and Additions to the Purification System

Item	Estimated Cost*
Six additional filters, plant interconnections and equipment at Dalecarlia	$ 626,000
New raw water intake and conduit to filter plant at Dalecarlia	306,000
New chemical building at Dalecarlia	1,301,000
Eighteen new filters at Dalecarlia	2,202,000
New flocculation-sedimentation basins at Dalecarlia	1,790,000
New 1-mil.gal. wash water reservoir at Dalecarlia	124,000
Utility relocations, connecting conduits and site improvements at Dalecarlia	458,000
Remodeling Georgetown reservoir	722,000
McMillan filter plant improvements to increase nominal capacity from 100 to 125 mgd.	409,000
	$7,938,000

* Based upon 1943 cost index of 290.

The above improvements will increase the nominal filtered water capacity of the purification system from 185 to 342 mgd., an amount estimated to be adequate to meet the total maximum daily demand until the year 2000.

Distribution Pumping Facilities

All purified water supplied to the high-service areas, including Arlington, is pumped. In the District of Columbia, there are two prime pumping stations—Dalecarlia and Bryant Street—and two auxiliary pumping stations—Anacostia and Fort Reno. The prime stations pump from the clear water basins into the transmission and distribution systems, with the distributing reservoirs acting as equalizers. Normal hydrostatic lift varies from 92 ft. to 288 ft. The Fort Reno auxiliary station takes suction from the third high-service reservoirs and pumps into the fourth high-service distribution system, or into elevated water tanks with an overflow elevation of 485.0 ft., as the demand requires. The Anacostia auxiliary station takes suction from the low-service trunk main system and pumps to the Anacostia first and second high-service distribution systems or to ground surface reservoirs and elevated storage tanks, as the demand requires. Hydrostatic lifts at the auxiliary stations vary from 60 to 237 ft. The installed and working capacities of the existing pumping stations are given in Table 7.

The total maximum daily demand in the year 2000 is estimated at 342 mgd., and, of this, 217 mgd. will be supplied by the Dalecarlia filter plant and 125 mgd. by the McMillan filter plant.

Abnormal demands will be met by the overload capacities of the plants.

To determine the pumping capacities required to meet the future demands of the high-service areas, including Arlington, a detailed study was made of the nominal and overload capacities of the Dalecarlia and McMillan filter plants that supply them and the physical characteristics of the transmission and distribution systems. The principal objective of this allocation was

TABLE 7

Capacity of Prime and Auxiliary Pumping Stations

Pumping Stations and Service Area	Motive Power	Hydrostatic Lift	Total Installed Capacity	Installed Capacity by Units	Maximum Working Output
		ft.	*mgd.*	*mgd.*	*mgd.*
PRIME STATIONS					
Dalecarlia					
First high	Electric	112	60	20–20–20	48
Second high	Electric	197	40	20–10–10	25
Third high	Electric	288	40	20–10–10	35
Arlington	Electric	288	10	10	12
TOTAL			150		120
Bryant Street					
First high	Electric	92	25	25	25
	Steam	69	50	30–20	46*
Second high	Electric	177	25	25	25
	Steam	150	15	15	12*
Third high	Electric	268	20	20	23
	Steam	277	10	10	10*
TOTAL			145		141
AUXILIARY STATIONS					
Fort Reno					
Fourth high	Electric	60	21.5	5–5–3½–3½–3–1½	18
Anacostia					
First high	Electric	113	23	10–8–5	19
Second high	Electric	237	13	6–5–2	11
TOTAL			57.5		48

* Head capacity insufficient to pump to reservoir levels that will maintain adequate pressures. Dependable pumping capacity in Bryant Street station is 73 mgd. in electrical equipment.

the anticipated population and water consumption increases in each area. As a result, the high-service water demands were allocated to the Dalecarlia and Bryant Street pumping stations, proportioned to their relative locations in the water supply system, the maintenance of optimum pressures and supply. A graphic picture of the allocation of pumping for the year 2000 is presented in Fig. 5.

At present, the low service is supplied by gravity flow, but, when the low-service distributing reservoir is

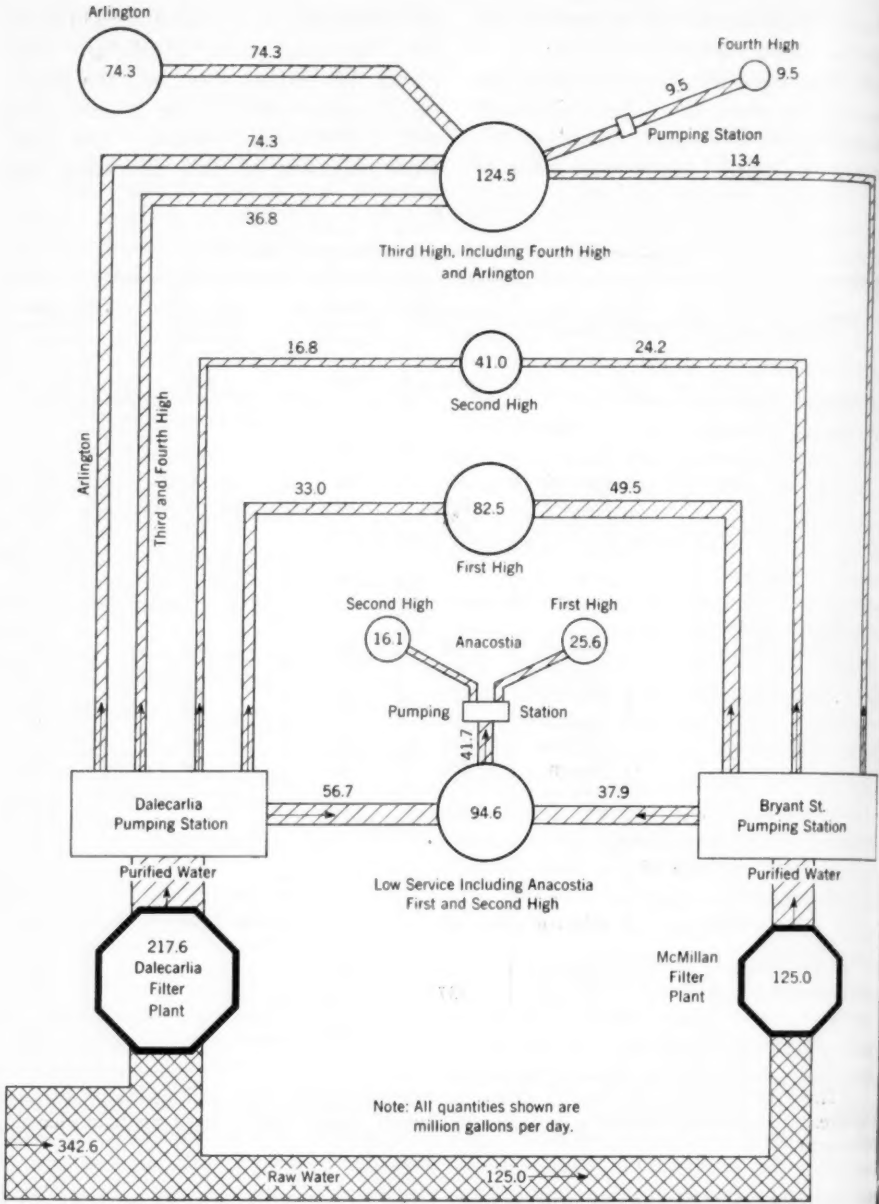


FIG. 5. Allocation of Pumping—Max. Day of Year 2000

constructed, water to all services will be pumped.

The total installed pumping capacities of the Dalecarlia and Bryant Street pumping stations for the year 2000 will be 450 mgd. and 220 mgd., respectively. The 450-mgd. pumping capacity at the Dalecarlia pumping sta-

tion includes 80 mgd. in pumping capacity to supply water to the Bryant Street pumping station through the crosstown inter-connecting main if the Washington city water tunnel should fail. These capacities provide not only for the normal maximum daily demand but, in addition, for reserve capacity against ordinary physical and operational failures and 25 per cent overload operation of the filter plants. The will be made to the two auxiliary stations. The existing pumping station at Fort Reno serving the fourth high area is located on the site of the future 40-mil.gal. third high reservoir and must be removed. A new station will be constructed, and, because only a limited quantity of elevated storage is practicable in the fourth high area, pumping equipment of 44-mgd. capacity—sufficient to follow hourly demands—will be installed. This station will take suction from the third high service reservoirs.

TABLE 8

Maximum Demand in Year 2000 and Proposed Pumping Capacities in Prime Stations

Water Service Areas	Population Served		Water Demand Maximum Day *mgd.*		Pumping Capacity with Reserve *mgd.*	
	From Dalecarlia	From McMillan	From Dalecarlia	From McMillan	From Dalecarlia	From Bryant Street
Low service and Anacostia high services	198,900	178,500	56.7	37.9	175*	70
First high	138,000	206,800	33.0	49.5	55	85
Second high	84,100	121,300	16.8	24.2	30	40
Third high	112,400	55,200	27.3	13.4		25
Fourth high	29,800	0	9.5	0		
Arlington, Va.	330,000	0	74.3	0		
Total third and fourth high and Arlington, Va.	472,200	55,200	111.1	13.4	190	
GRAND TOTAL	893,200	561,800	217.6	125.0	450	220

* Includes 80 mgd. pumping capacity to be used in case of failure of the Washington city water tunnel.

Dalecarlia pumping station will be of entirely new construction; the interior of the existing building at Bryant Street, however, will be altered and modified to meet the requirements of the new pumping equipment. The installed pumping capacities and populations served in the prime stations are shown in Table 8.

In addition to the proposed increase in pumping facilities of the two prime stations, improvements and additions

The Anacostia pumping station will be enlarged to 60-mgd. capacity to meet the anticipated future demands of both the Anacostia first and the Anacostia second high services. Storage in the second high area has to be limited because, topographically, only elevated storage can be obtained; hence, it is necessary to supplement the stor-

age with direct pumping to follow hourly demands.

The Anacostia station takes its suction water from the low-service trunk main system, and, to avoid low pressures in this area during periods of peak demands, the Anacostia first high reservoirs will be drawn upon heavily during the daytime. In order to restore the water thus drawn, high rates of nighttime or off-peak pumping will be required. This condition will be met by installing greater pump capacity for the first high service than otherwise would be necessary.

From the foregoing considerations and conditions, it is estimated that the total installed pumping capacities required to meet the maximum demands to the year 2000 will be 450 mgd. at Dalecarlia, 220 mgd. at Bryant Street, 44 mgd. at Fort Reno and 60 mgd. at Anacostia.

Storage Reservoirs

In the District of Columbia, reservoirs for the storage of purified water serve an important purpose in the safe and economical operation of the water system. Additional storage will be provided to meet the future increase in consumption, as the total existing distributing reservoir capacity is sufficient to meet only 12 hours of the maximum daily demand. Additional clear water basin capacity also will be provided at the Dalecarlia filter plant.

There is no uniform standard practice for setting the ratio between clear water basin capacity and the nominal filter plant capacity. Although it is true that minimum working storage capacity may be theoretically derived as a function of hourly demand and installed filtering and pumping capacity, the magnitude of the desire and the ability to provide for safeguarding the water supply with reserve storage are variable throughout the United States. After an analysis of the clear water basin capacities in many cities throughout the country, a minimum of 20 per cent was adopted for Washington as the relationship between clear water basin capacity and nominal filter plant capacity for the Dalecarlia filter plant. Planned extensions at this plant will increase its capacity from 85 mgd. to 217 mgd. Existing clear water basin capacity is 14.5 mil.gal., and, hence, by the criterion stated above, additional capacity of 30 mil.gal. will be required. No increase in clear water basin capacity at the McMillan filter plant is planned, as the existing capacity of 33 mil.gal. is considered sufficient.

Distributing reservoirs are of economic importance and also provide a factor of reserve water to insure against disaster. The hilly terrain of Washington is particularly favorable to the construction and use of ground surface storage reservoirs (Fig. 1). Security of supply and economy of operation are obtained: by permitting peak loads to be carried by the reservoirs, thus allowing the use of pumps of less capacity; by a constant pumping rate and uniform pressure, permitting operation with lower electrical demand charges; and, by the proper location of reservoirs, avoiding duplication of distributing mains.

The size of storage reservoirs in Washington is intended to serve two general purposes: to give sufficient working capacity for smooth and economical operation of the filter plants and pumping stations, and to provide sufficient reserve capacity for a reasonably continuous water supply during operational and physical failures in the supply system. Among the many

TABLE 9

Relation of Capacities of Storage Reservoirs to Daily Demands

Distributing Reservoirs	Daily Demand (Year 2000)		Existing Storage			Storage After Proposed Additions		
	Ann. Avg.	Max.	Capacity	Part of Ann. Avg. Day	Part of Max. Day	Capacity	Part of Ann. Avg. Day	Part of Max. Day
	mgd.	*mgd.*	*mil.gal.*	%	%	*mil.gal.*	%	%
Low-service area including Anacostia first and second high areas								
Fort Stanton (old)			3.0					
Fort Stanton (new)			10.0					
Good Hope elevated			0.5					
Fort Dupont elevated			2.0					
	64.9	94.6	15.5	23.9	16.4	55.5	85.5	57.8
First high-service area								
Old first high			14.5					
Soldiers' Home			15.0					
	58.7	82.5	29.5	50.3	35.8	49.5	84.3	60.0
Second high-service area								
Second high reservoir	28.9	41.0	14.6	50.5	35.6	26.6	92.0	64.8
Third high-service area, including fourth high area and Arlington, Va.								
Fort Reno (old)			4.7					
Fort Reno (new)			5.4					
Fort Reno elevated (old)			0.08					
Fort Reno elevated (new)			0.16					
Minors Hill			6.0					
Lyonhurst			1.5					
Lyonhurst elevated			0.25					
Fort Barnard elevated			0.5					
Fairlington elevated			0.5					
	72.5	124.5	19.09	26.3	15.3	72.39	100.0	58.1
TOTAL	225.0	342.6	78.69	35.0	23.0	203.99	90.7	59.5

communities in the United States which have distribution reservoir capacities greater than the average daily demand are such representative cities as Pittsburgh, Cincinnati, St. Louis, Cleveland and Baltimore.

There are nine ground surface reservoirs and seven elevated storage tanks in the existing water distribution systems of the District of Columbia and Arlington, Va. Their capacities are shown in Table 9.

The water consumption demand of Washington and Arlington varies hourly during any day. On maximum days, the actual hourly rate will exceed the 24-hour average hourly rate for a period of about 15 hours. This

means that, when the filter plants and pumping stations are operating at their nominal capacities for the 24-hour day, there will be about 15 hours when the demand will exceed the nominal capacity of the plants to produce water. This excess demand will amount to about 15 per cent of the maximum daily demand, and, in the low-service area and the areas supplied from the third high distributing reservoirs, the excess hourly demand will amount to about 20 per cent of the maximum daily demand. This excess will be supplied from the available working storage in the distributing reservoirs of the system.

As a result of the importance of the water supply of the national capital and its environs, the argument in favor of adequate storage to assure some safeguards during emergencies and the absence of a specific rule having general application, there is planned for each service area a total working and reserve filtered water storage capacity equal to between 75 per cent and 100 per cent of the consumption demand for the annual average day. The expenditure to obtain elevated storage is unjustifiably large, however, and, if adequate sites for surface storage are unobtainable, this principle will be modified, without incurring any specific hazards, to meet the financial requirements deemed applicable to Washington.

The existing distributing reservoir capacity will be increased considerably to meet the water demands to the year 2000. Additional reservoir capacity will be installed in all water service areas with the exception of the Anacostia second high area, where the existing elevated storage of 2.5 mil.gal. augmented with auxiliary pumping to meet peak demands is deemed sufficient.

The comparison between the ratios of the existing and future reservoir storage capacities to the average and maximum daily demands for the year 2000 is given in Table 9. Construction of distributing reservoirs to secure the increased capacity shown in this table is necessary to develop the full capacities of the filter plants and pumping stations, to make an adequate quantity of water at the proper pressure immediately available to consumers at peak hourly demands and to assure reasonable safety throughout the water system by providing reserve storage in addition to the working storage already existing.

Mains and Auxiliary Facilities

The distribution system within the District of Columbia is divided into seven water service areas, the ground elevations of which range from 10 to 420 ft. above mean sea level. Besides pumping stations and purified water reservoirs, the system includes approximately 1,066 miles of water mains, 18,700 valves and 7,119 fire hydrants. At present, the city is about 94 per cent metered. Distribution pipelines within the District of Columbia range in diameters up to 75 in. and are largely of cast iron, steel and concrete; a small amount are of asbestos cement. In addition, water is supplied to federal agencies outside the city through a distribution system consisting of 29.3 miles of cast-iron and steel pipe ranging in diameters up to 30 in. with appurtenant valves, fire hydrants and meters. Water is supplied also to Arlington County directly from the Dalecarlia filter plant through a 24-in. and a 36-in. main. The county operates and maintains a complete distribution system, consisting of pumping stations, surface reservoirs, elevated

tanks and 239.8 miles of cast-iron, steel and concrete pipelines ranging in diameters up to 36 in., with appurtenant valves, fire hydrants and customers' meters.

In fixing the sizes of future trunk water mains required to supply the basic areas, the flow capacity was established according to the population, character of service and fire demand. This procedure is followed, in general, except in highly developed areas, particularly in the low-service and first high-service areas, where conditions of practical saturation have been reached and the sizes of needed additional trunk mains are known from experience and from hydraulic investigations.

Trunk mains will be sized on the basis of maximum hourly flows. A study of the records of hourly peak demands indicates that the ratio of maximum hourly demand to the average hourly demand on maximum days varies between the several service areas. Considering the city as a whole, the ratio is 1.6. This ratio in the low service on the west side of the Anacostia River has been assumed, for trunk main studies, as 1.5; in the first, second and third high-service areas, as 1.6; on the fourth high service, as 2.0; in the Anacostia low service and first high service, as 1.6; and, in the Anacostia second high service, as 2.0.

Minimum pressure sought to be maintained within the various areas under maximum hour conditions is 35 psi. The criteria for fire flow of the National Board of Fire Underwriters (1) have been considered carefully and future distribution trunk mains are to be sized accordingly.

In determining the sizes of future trunk water mains 20 in. and greater in diameter, a Williams and Hazen flow coefficient of 120 has been employed. This contemplates the use of pipe having an interior surface equivalent to concrete pipe, cast-iron pipe with cement lining, or steel pipe with bituminous-enamel lining. Although straight runs of pipe of these types actually will show a coefficient in the neighborhood of 135 to 145, the lower figure has been selected in order that there may be an appropriate compensation for loss resulting from bends, valves, tees and sudden changes in velocity through changes in the size of the mains. The pipe sizes selected have been such as to limit loss of head at maximum flow rates to from 1½ to 2 ft. per 1,000 ft. of main.

The principal feature in the future expansion of the distribution system is a crosstown inter-connecting trunk main connecting the Dalecarlia pumping station with the Bryant Street pumping station, thereby bringing about the integration of the Dalecarlia and McMillan filter plants. This inter-connecting main will be provided with several connections into the low-service area and also will tie into the proposed southeast relief trunk main which will convey water to the low-service area in Anacostia and to the Anacostia high-lift pumping station. The crosstown main will be designed with sufficient capacity over its normal requirements to supply water from Dalecarlia to the Bryant Street station if the Washington city water tunnel, which conveys settled, alum-treated raw water to the McMillan filter plant, should fail. Together, the crosstown inter-connecting main and the southeast relief trunk main system will be about 13.4 miles long, ranging in diameter from 30 to 78 in. and, as shown on Fig. 6, will tie the low-service and Anacostia areas to both the Dalecarlia and McMillan filter plants. Pressures

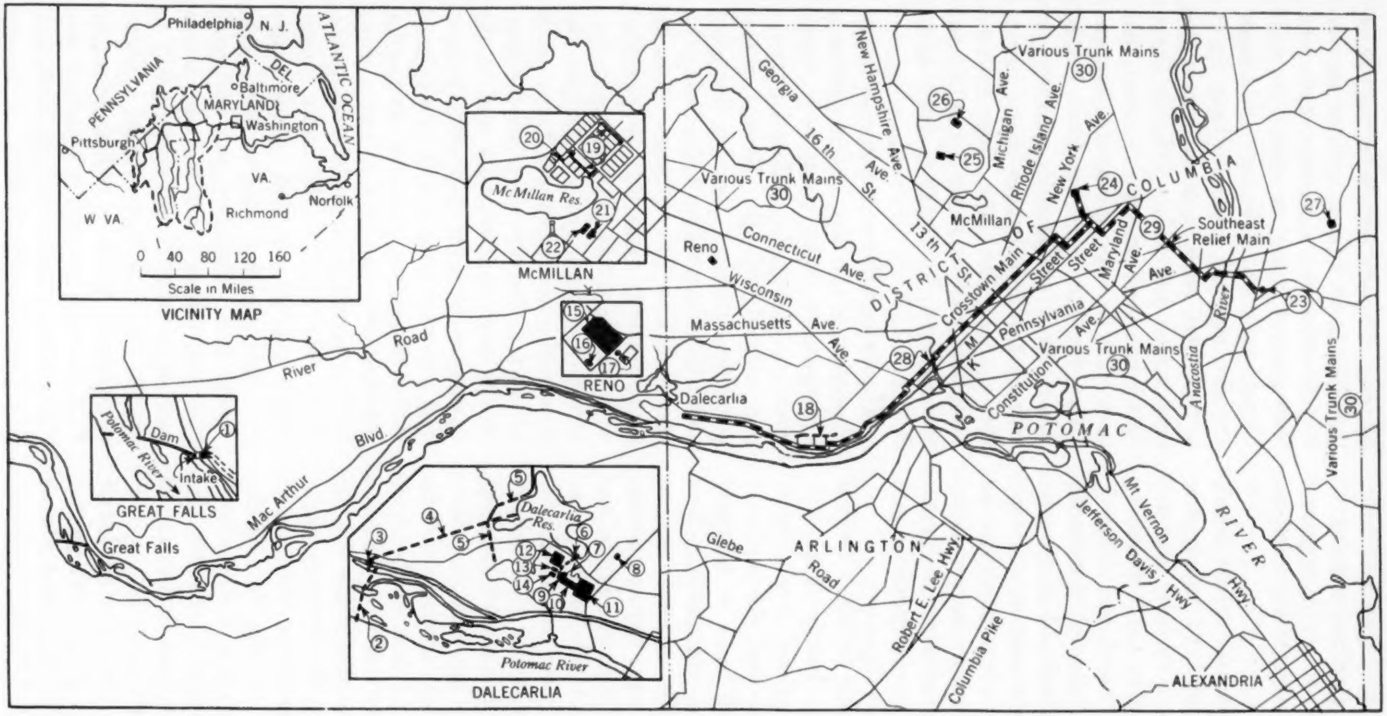


FIG. 6. Map of System Improvements

1. Great Falls intake
2. Little Falls dam
3. L.F. pumping sta.
4. L.F. rising tunnel
5. Dalecarlia Res. flood improvements
6. Dal. Filter Plant, intake
7. Dal. F.P., 6 addnl. filters
8. Dal. F.P. wash water reservoir
9. Dal. F.P. chemical bldg.
10. Dal. F.P. 18 addnl. filters
11. Dal. F.P. mixing & sedimentation basins
12. Dal. F.P. clear water basin
13. Dal. F.P. pumping sta.
14. Dal. F.P. storehouse
15. 40-mil.gal. reservoir
16. 4th high pumping sta.
17. 1½-mil.gal. storage tank
18. Georgetown Res. improvements
19. McMillan Filter Plant improvements
20. McM. F.P., storehouse
21. Bryant St. pumping sta.
22. Bry. St. storehouse
23. Anacostia pumping sta.
24. 25-mil.gal. low-service res., Brentwood
25. 20 mil.gal. 1st high res., Soldiers' Home
26. 12-mil.gal. 2nd high res.
27. 15-mil.gal. Anacostia 1st high res.
28. 78-in. crosstown main
29. Southeast relief main
30. Various trunk mains

in the low-service area both east and west of the Anacostia River will be increased appreciably by the construction of the crosstown main.

As development in the high-service areas takes place, increasing water demands will require expanded trunk main and distribution system pipelines, especially in the peripheral zones adjacent to the central part of the city, including the third high, fourth high and the first and second high Anacostia areas.

Examination of distribution facilities, in the light of water requirements

TABLE 10

Trunk Main Improvements

Service Area	Size Range	Total Length
	in.	*ft.*
Low service	24–36	24,100
First high	20–48	82,900
Second high	20–36	69,200
Third high	20–48	43,400
Fourth high	20–24	9,900
Anacostia first high	20–30	36,300
Anacostia second high	30–42	14,000
TOTAL		279,800

expected to develop from the present time to the year 2000, indicates the necessity for extensions and reinforcement within all of the various water service areas. Present transmission mains lend themselves admirably to caring for future needs and future transmission mains will consist primarily of a continuation of the existing systems. New elements to be added to the distribution system will be the crosstown inter-connecting main, the southeast relief trunk main system, and the 48-in. main from the Dalecarlia pumping station to the 40-mil. gal. reservoir at Fort Reno serving the third high area. The adjustment of the limits of the territories supplied by the various water services to conform as nearly as practicable to their designed contour boundaries will round out the distribution system. In some cases, certain mains will be changed over for use in the next higher or lower service area as operating conditions require. Many of the pipeline extensions will result in improvement of territory now largely undeveloped which, during the period to the year 2000, will reach virtual population saturation under existing zoning ordinances.

TABLE 11

Secondary Main Improvements

Service Area	Size Range	Total Length
	in.	*ft.*
First high	8–16	120,200
Second high	8–12	169,900
Third high	8–16	215,300
Fourth high	8–16	69,500
Anacostia first high	8–16	193,500
Anacostia second high	8–16	157,900
TOTAL		926,300

A summary of proposed trunk main improvements within the several service areas exclusive of the crosstown and southeast relief systems described above is shown in Table 10.

In addition to the larger size trunk mains shown above, numerous secondary feeders and grid water mains will be necessary as development proceeds in the various service areas. Full utilization of the trunk main system is dependent upon connection with secondary lines ranging in size from 8 to 16 in. The estimated sizes and quantities of secondary main improvements are shown in Table 11.

The Washington water system is complex, involving many miles of supply and distribution mains serving seven different service areas, many pumping stations, numerous reservoirs and two large filtration plants. For this reason, certain auxiliary facilities of a general nature that, to a large extent, apply to the water system as a whole will be constructed.

Telemetric communication facilities for rates of flow, reservoir levels and pressures in the various areas will be installed so that conditions in all parts of the water system may be known continuously at the operating centers. Some of these facilities are now in use to a limited extent, but further improvements will be made to serve the expanded system.

Application of chlorine at the Dalecarlia and McMillan filtration plants is the standard practice, and chlorine residuals are present to a certain degree throughout the distribution system. As the Anacostia region is the most remote from the source of chlorination, residuals frequently disappear entirely before water has reached the extremities of this service area. Additional chlorination facilities will be provided to give positive residuals throughout the system, for the public health authorities are advocating the maintenance of more definite chlorine residuals throughout the distribution system, notwithstanding the absence of epidemiological experience in Washington. A preliminary study indicates that the introduction of chlorine at the filtration plants, in quantities that would give the desired residuals in the system extremities, would require excessively large quantities of chlorine. The residuals near the filtration plants would, therefore, be objectionably high, and, for the most part, would be dissipated without corresponding benefit where required. Accordingly, the construction of supplementary chlorination facilities for the purpose of boosting the chlorine residuals in remote parts of the system is scheduled.

Examination of the active water services during the war years indicated that there was one service to each 6.5 persons resident in the District of Columbia. Because the city at that time had a highly congested hotel, rooming house and military population, this ratio is considered too high to be used when forecasting ultimate normal demands. The ratio for the year 1940 was approximately 5.9 persons to each active service connection, and this ratio has been employed for the purpose of future meter forecasting. The population increase between the present time and the year 1990 would indicate the necessity of installing approximately 55,000 meters, to which would be added about 8,000 flat rate accounts that will be metered as soon as practicable, making a total of 63,000 new meter installations.

As the distribution system is extended into undeveloped areas, fire hydrants will be installed in general accordance with the Fire Underwriters' requirements.

Additional meter repair, garage, shop and storehouse facilities will be provided at the Dalecarlia and McMillan filter plants and at the Bryant Street pumping station, as the existing facilities already have become inadequate for present-day needs.

Provision has been made for rearrangements that will be required to maintain continuity of service during the period of construction of the new system facilities of pumping, piping inter-connections and other appurtenances. Some of these installations

will be permanent; others will have to be temporary, to serve a particular construction need.

Construction Program

The comprehensive plan and program described in this paper provides for the construction of additions and improvements to the water system of the District of Columbia to meet safely and adequately the anticipated future water demands during the 53-year period ending in 2000. In general, the plan provides for all the necessary facilities for the collection of raw water, pre-treatment and sedimentation, filtration, corrective chemical treatment and sterilization, pumping, transmission, storage and final distribution to bring all elements of the present and proposed works up to a uniform standard of modern water works practice.

The allocation of water quantities to the several service areas is based upon the anticipated distribution of water users to the year 2000. Because of the difficulties involved in estimating the future distribution of consumers, the comprehensive plan must be flexible, particularly in its pumping equipment, transmission mains, trunk mains, and, to some extent, distributing reservoirs. The logical approach lies in periodic analyses of the actual water demands of the future, so that the comprehensive plan may be modified as nearly as possible to encompass the actual conditions as they materialize. Figure 7 lists the various items in the comprehensive plan for the expansion of the water system of the District of Columbia and environs, the cost of each item, the year of construction, and the total yearly expenditure.

The time and sequence of construction follow generally the need for facilities as determined by the increasing demand for water. Those facilities which will relieve the overloading of the system, as indicated on Fig. 4, will be built first. This increment in capacity then can be utilized to permit subsequent construction without system interruption and, at the same time, will hold the annual cost of the construction program to a smaller and more uniform expenditure.

Estimates of cost of the structures involved were prepared after research into costs of similar structures in both the local water works system and those of other comparable cities. These costs were adjusted to a common base, using the *Engineering News-Record* cost index base of 1913. The cost index used was 290, which prevailed in 1943, the time when many of the estimates were made. Since then, the *Engineering News-Record* cost index has increased to 360, as of September 1946. Although the estimates made in 1943 are considered reasonably correct for that time, the actual costs necessarily will vary with the time of construction. If the cost index at any future date is different, it will be feasible at such a future time to arrive at the estimated cost of the facilities by the use of the proper cost index ratio.

The construction program over the 1947–1990 period is divided into four stages. Most of the basic expansion and improvement work, however, is to be completed by 1961. The facilities that are most urgently needed, and that will be provided during the first construction stage, are the three major pumping stations: Bryant Street, Dalecarlia and Anacostia; the improvements to the McMillan filter plant increasing the nominal capacity from 100 to 125 mgd.; at Dalecarlia, the six additional filters totaling 24 mgd., the

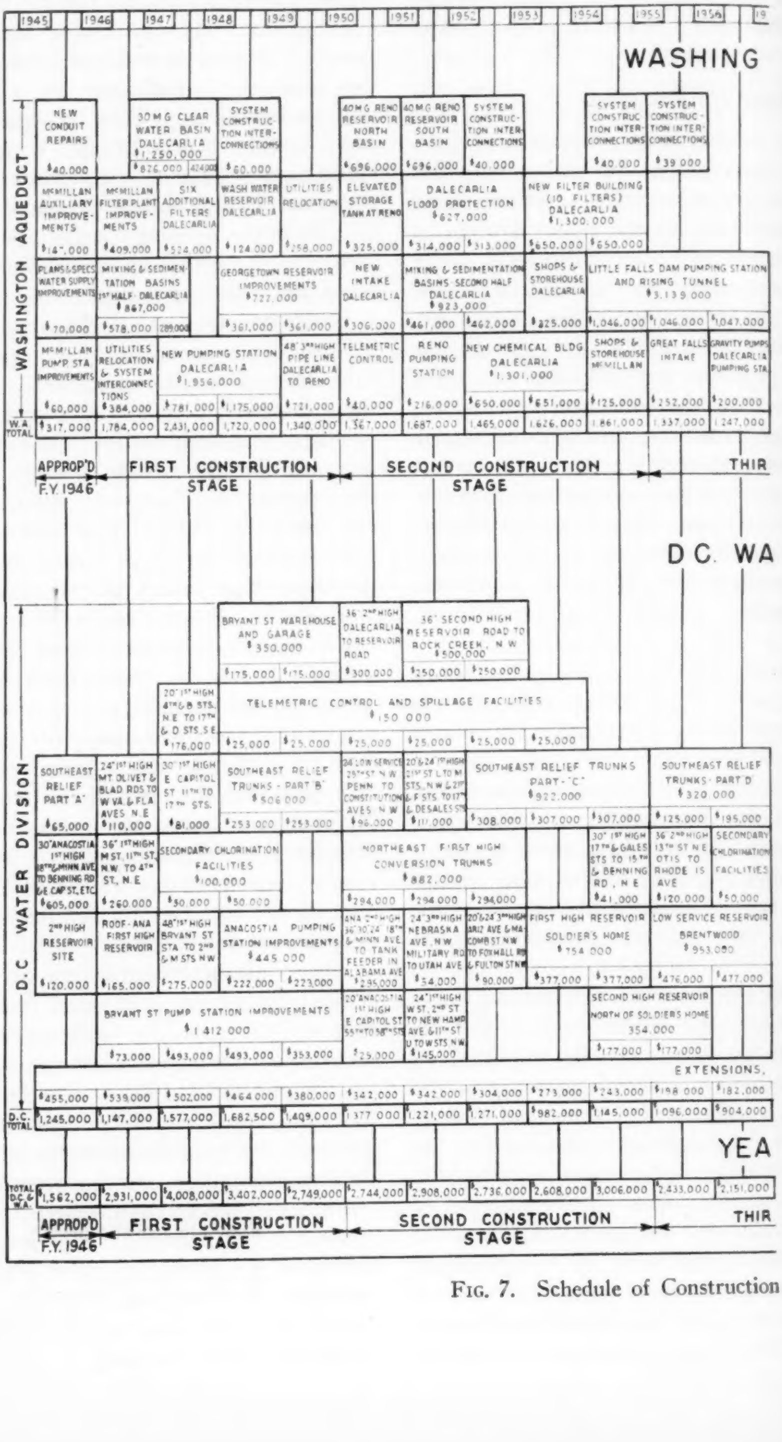
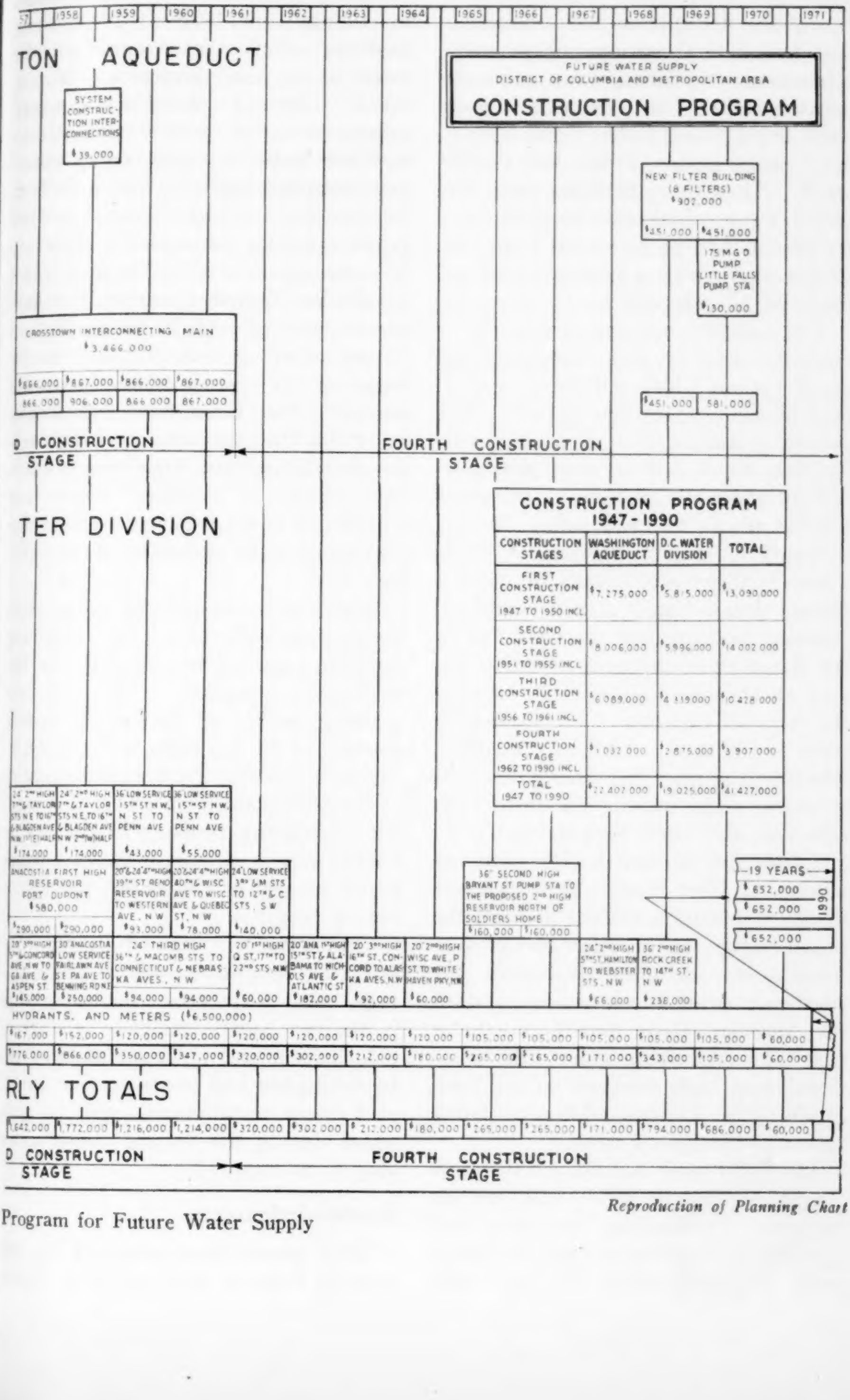


FIG. 7. Schedule of Construction



Reproduction of Planning Chart

Program for Future Water Supply

30-mil.gal. clear water basin, the first half of the flocculation and sedimentation basins, and various plant interconnections; the Georgetown reservoir improvements; a part of the southeast relief trunk main system; and certain other trunk mains within the service areas. After these facilities are completed, the nominal capacity of the water system will be increased from 185 to 234 mgd., with a safe overload capacity of 10 per cent.

The second construction stage is to meet the capacity requirements of the water system which will have eventuated by the end of this period. The supply system will be provided with facilities which will increase the nominal filtered water capacity of the purification plants to meet safely the anticipated maximum demand of 1955. These facilities will consist of 10 new filters, intake and screen building, chemical building and the west half of the flocculation and sedimentation basins at Dalecarlia; they will increase the nominal capacity of the Dalecarlia plant to 169 mgd. Additional facilities which will be provided during this construction stage include the third high reservoir, the fourth high elevated storage tank and the fourth high pumping station at Fort Reno; improvements in the distribution system to meet the broad changes in the water service area boundaries, including primarily the southeast relief trunk mains west of the Anacostia River and the northeast first high conversion mains; the additional first high reservoir at Soldiers' Home; and various additional trunk mains and auxiliary items.

The first and second construction stages, ending in 1955, will provide sufficient purification, pumping and distribution facilities to meet the maximum daily demand to the year 1970.

During the third construction stage, extending from 1955 to 1961, those facilities which are essential to the safety of the water system will be provided. The major items of the supply system consist of the new intake works at Great Falls; the dam, rising tunnel and pumping station at Little Falls to increase the raw water supply; and the crosstown inter-connecting main which will enable the Dalecarlia filter plant to deliver filtered water to the low-service area as well as to the Bryant Street pumping station. The major items of the distribution system will comprise the Brentwood low-service reservoir, the Anacostia first high reservoir, the second high reservoir in the vicinity of Soldiers' Home and completion of the principal trunk mains serving or to be served by these facilities.

The fourth construction stage comprises practically all of the remaining facilities required to complete the improvement program. The only remaining facility of the supply system consists of the last eight filters at Dalecarlia, increasing the nominal capacity of the filter plant from 169 to 217 mgd. The remaining projects in the distribution system comprise various trunk mains needed to round out the distribution program.

Other items contained in the construction program—such as shop, garage and warehouse facilities; fire hydrants; small mains; telemetric, chlorination and spillage facilities between higher and lower service areas; and meter installations—will be provided during the several construction stages, as required.

Acknowledgments

This paper was prepared by the authors from a comprehensive report

entitled "Adequate Future Water Supply for the District of Columbia and Metropolitan Area," which they took an active part in preparing, and which was submitted to the Congress in February 1946. The report was prepared under the direct supervision of Edwin A. Schmitt with the assistance of members of his staff, including Dewey M. Radcliffe, Chief of Operations; Philip O. Macqueen, Chief of Eng.; Otto D. Voigt, Head of Planning; and Carl J. Lauter, Chem. Engr. Representatives of the District of Columbia who collaborated and assisted in the preparation of the joint report were the late John Blake Gordon, then Director of San. Eng., and Harold A. Kemp, his successor; and members of the Water Division staff, including Humphrey Beckett, former Supt., and David V. Auld, his successor; Paul Lanham, Sr. Engr. in charge of Operations and Maintenance; and Roy L. Orndorff, Asst. Supt. in charge of Design and Construction. The firm of Greeley and Hansen, consulting engineers of Chicago, was retained to collaborate in the preparation of the report and to make a final review.

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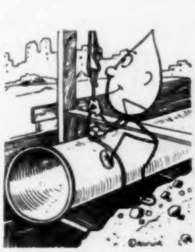

The Role of Algae in Corrosion

By Henry C. Myers

A paper presented on Oct. 23, 1946, at the California Section Meeting, San Francisco, Calif., by Henry C. Myers, San. Engr., San Diego Bay Div., California Water & Telephone Co., National City, Calif.

CORROSION due to the prolific growth of *Oscillatoria*, a member of the blue-green or *Myxophyceae* class of algae, occurred under quite unusual conditions in open steel tanks of the South Basin Softening Plant of the California Water & Telephone Co., located in the lower Tia Juana Valley near the Mexican border.

Chemistry of Corrosion

The theoretical manner in which corrosion caused by algae progresses may be represented as follows:

$$H_2O \rightleftharpoons H^+ + OH^- \quad \ldots\ldots (1)$$
$$2H^+ + 2e = 2H^0 \quad \ldots\ldots (2)$$
$$Fe^0 - 2e = Fe^{++} \quad \ldots\ldots (3)$$
$$2H^+ + Fe^0 = 2H^0 + Fe^{++} \quad \ldots (4)$$

Water is always dissociated, as in Eq. 1, thereby supplying hydrogen ions. Equations 2 and 3 are dependent upon each other. In accepting electrons, hydrogen ions are reduced to hydrogen atoms with a consequent valence of 0. Metallic iron yields these electrons, and in so doing it is oxidized to the ferrous state, with a valence of 2. Since Eq. 2 and 3 are so related to each other, they may be added. This gives the over-all reaction expressed in Eq. 4.

The atomic hydrogen which is produced is sometimes referred to as nascent hydrogen. It is relatively unstable and tends to unite with other atoms of its own kind to form stable molecules, as: $2H \rightleftharpoons H_2$; or it may unite with some other available element and produce a compound. If H_2 is formed, the hydrogen tends to coat the metal and the metal is then said to be polarized. No further corrosion is possible unless this film of hydrogen is removed; that is, the metal must become depolarized.

Removal of the film of hydrogen may be accomplished mechanically; atomic hydrogen, however, may be removed by naturally occurring chemical methods without ever forming molecular hydrogen. Oxygen is liberated in abundance by algae as a waste product through the life process of photosynthesis, and atomic hydrogen will unite with this oxygen instead of producing hydrogen molecules. This results in the formation of water as expressed in the following equation.

$$2H + \tfrac{1}{2}O_2 = H_2O \quad \ldots\ldots (5)$$

In view of the facts that hydrogen is eliminated as rapidly as it is produced, and that Eq. 2 and 3 are dependent upon each other, iron can continuously go into solution as ferrous iron, even in waters which are not considered to be aggressive. This action causes serious pitting of steel wherever *Oscillatoria* grows near unprotected steel. The pits are bright and clean because the iron goes into solu-

tion instead of producing some covering compound such as, for example, an oxide or sulfide. *Oscillatoria* is specifically mentioned because it is this alga that has been studied at South Basin, but any species of algae which grows in close proximity to steel should produce the same undesirable results. It is to be remembered that the presence of dissolved oxygen, regardless of source, is a factor which always aids corrosion.

Growth of Algae

The water being softened at South Basin comes from wells and is free from dissolved oxygen. It might be thought that the water would be more favorable for algal growth as it enters the plant, because of an abundance of carbon dioxide and a nearly neutral pH of 7.3. *Oscillatoria,* however, does not grow in the untreated water, nor in the open storage reservoir which contains the filtered plant effluent. Other forms of algae grow in the untreated water, but never *Oscillatoria.* This organism grows only in the open secondary mixing tanks, sedimentation tank and filters. The water in these places has a pH of 9.4–9.5, a phenolphthalein alkalinity of about 35 ppm. and a methyl orange alkalinity of about 130 ppm. Such a water should neither favor the rapid growth of algae nor cause undue corrosion, yet corrosion went on at an alarming rate and it does not seem that *Oscillatoria* could have had a more favorable environment.

The algal filaments spread out rapidly, forming dense mats on the sides of the tanks, the filter walls and even the filter surface. At intervals of ten days to two weeks it was necessary to scrape or wash down these walls. No paint could last long under such rough treatment, and, where paint was removed, pitting occurred. Clean holes deep into the metal were observed. Pitting was at its worst near the top of the tanks and did not occur near the bottom, where lack of sufficient sunlight prevented algal growth. In fact, the bottom 5 or 6 ft. of steel is in as good condition now as it was when the plant was built ten years ago.

Attempts at Control

Attempts were made to control *Oscillatoria,* not only because it was believed to be the cause of corrosion but also because of the unsightly appearance of such growths. For some time small quantities of copper sulfate were continually added to the sedimentation tank. With water such as is encountered at this stage of softening, however, copper does not stay in solution long enough to be effective. Chlorination would eliminate algae, but an excessive quantity of chlorine would be required under operating conditions at this plant. The finished water is chlorinated upon leaving the storage reservoir and, as the water entering the plant is of excellent bacterial quality, it was believed that chlorination for the control of algae was not the proper procedure. The use of activated carbon had been suggested as a means of controlling algae by shutting out the sunlight, but such a practice would be expensive and the results questionable in a plant similar to South Basin.

The simplest, least expensive and most effective method of control is to shut out the sunlight and deprive the algae of the source of energy which all plants containing chlorophyll require for their very existence. This can be most economically done by covering the tanks. Although the covering of sedimentation and mixing tanks

is not a common practice, it was decided to cover those at South Basin. At once *Oscillatoria* ceased to grow, and the unstable water began to deposit a layer of calcium carbonate on the steel. With this coating it has been found unnecessary to paint the inside of the tanks. Previously, the calcium carbonate deposits were so intermingled with filaments of *Oscillatoria* that the resulting deposit was not effective, and was removed with the algae when the tanks were cleaned. Since the tanks were covered there has been no further corrosion or pitting.

The calcium carbonate coating is quite soft, except in certain areas. In the region where by-passed untreated water is blended with the overtreated water (split treatment method), there forms a very hard crust which increases in thickness quite rapidly and has to be removed frequently. The same effect is found where the water falls over the lip of the sedimentation tank. Elsewhere a soft coating builds up in thickness until its own weight causes it to crumble from the walls. Continual formation of calcium carbonate, however, keeps an intact coating on the surfaces.

When the tanks were covered, one of the carpenters dropped his saw in the sedimentation tank. The saw was recovered a year later, when the tank was drained for inspection, in the same good condition as when it had been lost. The teeth had not lost their sharpness, and there were no pits nor signs of rust.

The clarification tank, in which a pH of 10.6 and about 50 ppm. of caustic alkalinity is maintained, is unpainted below the water line. No growths of any nature occur in this tank. Pitting and corrosion are non-existent under these conditions.

It has been the experience at South Basin that internal corrosion of tanks is not always eliminated by a coating of paint. This is due, at least in part, to the failure of paint in many small and scattered areas. The metal is then left exposed to any depolarizing agent which may be present. Where algal growths are encountered, the liberated oxygen acts as a powerful depolarizer. Corrosion under such conditions can be reduced by covering tanks containing either untreated water of a low pH or the high pH water found in softening plants.

Reduction of Municipal Taxes and the Support of Public Services

By Carl H. Chatters

A contribution to the Journal by Carl H. Chatters, Comptroller, The Port of New York Authority, New York. This paper was presented on Jan. 17, 1947 at the Eighth Biennial General Assembly of the States at Chicago.

FEW municipalities can reduce their taxes. A decreasing proportion will be able to finance the growing list of public services and social welfare payments. This inability to reduce taxes and to support services results generally from an increasing level of costs, a growing number of public services—primarily in the social welfare group—and a spotty and illogical distribution of the total public revenue.

First, it should be observed that the number of public services required of local governments has grown in number and cost, particularly since 1920, and that the sources and amount of local revenues have been comparatively constant, whereas state revenues have been increasing rapidly, and the number of state functions has not appreciably increased. Between 1926 and 1942 the states increased their tax yields thirteen times as rapidly as localities (1). If the period were extended to 1946 the discrepancy would be even greater.

Services of Government

The quantity and quality of public services determine the amount of money really needed by a municipality. For several years the author has been working on "An Inventory of Governmental Activities in the United States." It is shocking even to one who has lived intimately with government all his life. It is shocking, not in the sense of being bad, but because it shows so clearly how much government does, and how much more it does now than it did earlier in the century. For instance, in the list of public service enterprises operated by municipalities may be found railroads, street railways, bus lines, electric power plants, water works plants, gas plants, liquor stores, airports, ferries, bus terminals, markets, grain elevators, abattoirs, cemeteries and crematories, broadcasting stations, telephone systems, and ports and harbors. But these might all be self-supporting, so look at another list. Local police departments now prevent and investigate crimes, keep custody of their prisoners, maintain identification records and crime statistics, have extensive communication systems, supervise morals through suppression of prostitution, liquor control, narcotic control and the regulation of dance halls and amusement places. The police also maintain traffic signals, patrol city streets, inspect motor vehicles and conduct drivers' examinations.

The protective inspection services of a municipality are numerous, and their

necessity has been well justified by recent fires and other tragedies. There are municipal building inspection, plumbing inspection, electrical inspection, gas inspection, boiler inspection, elevator inspection, inspection of weights and measures, smoke inspection and numerous health inspections. Local services related to public health, social services, hospitals and public recreation could be listed at even greater length. Then there are the two areas in which expenditures have been so great: highways or streets, and public education. This catalog grows tiresome, so it will end with two pertinent observations: (1) that the states have also increased their activities but in many less fields; and (2) that the increased number and cost of services tend to make the small and local areas of government much more inadequate for performing these services and for financing them.

Let us look again at the questions under discussion: "Can taxes be reduced?" and "To what extent can municipalities support their public services?" Assume that municipalities are required to carry on only those services they had in 1926. Assume also that these services are performed as honestly and economically as possible. How can municipalities spend less for these services when salaries and wages are so much higher, when the consumers price index (August 1946) was 143.7 per cent of the 1935–1939 average, the retail price index (August 1946) 159.8 per cent of the 1935–1939 average and construction costs from 50 to 100 per cent higher than they were in 1940?

Several general lines of direct attack can be used to reduce municipal taxes. Taxes *can* be cut without reducing expenditures. That is a good old political trick with which everyone is familiar. But it does not help in the long run. Next, taxes can be reduced by doing nothing new or constructive, by failure to maintain present facilities or to build new ones. However, genuine tax reduction is possible in many municipalities by at least three devices. First, genuine economy will be aided by removal of political deadheads or other incompetents. They are a material factor in only a few places. Second, greater state grants-in-aid will reduce local taxes if properly administered. Otherwise they stimulate and increase municipal spending. Third, great savings may be made by proper arrangement of the areas for collecting taxes and conducting public services and by other changes in the form of government areas and administration.

The factors which would aid municipalities to support their public services or to reduce local taxes are divided here into two groups: what the municipalities can do and what the states can do.

What Municipalities Can Do

Municipalities can act independently in many ways to improve their ability to support their public services:

1. A consistently balanced budget over a long period of years is the best guarantee that municipality will be in the best position to meet its obligations. But there must be included as expenditures all vital maintenance, all necessary capital improvements and actual payment currently of all obligations incurred.

2. Recognize the high cost of municipal debt. Avoid it where possible. Debt fixes municipal costs for a generation and makes reduction of taxes or the increase of services impossible. The greatest municipal waste may be

found in this area. Sometimes it comes through inability to sell bonds properly. At other times bonds are sold when it is unnecessary to do so.

3. Municipalities can now and in the future support *some* services but not *all* services. They can support *more* services when their debts are low and the income of the community is high. They cannot be expected to support from local funds services which spread unequal burdens, such as social services, or services which increase out of proportion to municipal income, such as unemployment relief, education, unusual construction projects and many of the newer social services.

4. Civil service and good personnel methods must be strengthened in order to get and keep qualified persons on the payroll. The rights of permanent workers should be protected, and adequate provisions for retirement at superannuation should be made. The salaries of the top personnel in government should be increased substantially in order to attract and keep capable practical administrators and technicians. But the retention of incompetents by any means is just as deadening as political interference is demoralizing.

5. Municipalities need a more flexible and a more dependable revenue system, so that their revenues will increase with the price level or with added services and will not contract too greatly in times of depression. The states and the national government should be prepared to finance or take over the services which expand unduly or supplement the revenues which shrink where they are most needed. Remember that local taxes generally are tied to an inflexible base, whereas state and federal taxes, primarily on incomes and sales, automatically give more revenue as prices and incomes rise. Probably the states get less tax complaints than municipalities because the latter must increase tax rates to get more money, whereas the states, without a change in the rate of the income tax or sales tax, just sit by and watch the money roll in.

6. Municipalities that are homogeneous, that grow gradually, that build community wealth as population increases, that are not surrounded by parasitical communities, have few troubles and generally can support their public services. On the other hand, rapidly growing municipalities that must finance improvements by debt, municipalities that construct uneconomic facilities, school districts that share unequally and unfairly in revenues (such as those derived from railroad property in Illinois), municipalities surrounded by small communities which draw on the central city for many services but contribute little or nothing, and the very large cities from which the state siphons off income and returns too little, cannot now and never will be able under similar conditions to finance their services.

7. Municipalities should endeavor to strengthen their local revenues by every means which is legally, socially and economically sound. But the failure to do this does not give the state an excuse to impose unfair burdens by new services, nor does it give the state the right to absorb an unfair share of the total public revenue. The failure of municipalities generally to handle their financial affairs as well as they might, however, does give the state the excuse for interference in local affairs.

What the States Can Do

State governments can do many things which will make it possible for

the municipalities to work out their own financial problems or which will remove municipal financial burdens.

1. The states can and should refrain from granting exemption from real property taxes and expect the municipalities to carry the entire burden of the exemptions. Exemptions granted pursuant to state law affect an unbelievably large part of total property, and the exemptions are unevenly distributed throughout the state. The question should be examined and most exemptions removed or modified.

2. The states should finance, in part, those municipal expenditures which are not constant, which are distributed unevenly over the state, which result from acts of God or national economic disasters, or which cannot be fairly borne by the areas of government as presently constituted. In these categories is included, among other expenses, a great share of unemployment relief costs, funds to equalize educational costs within a state, removal of unusual snows in cities, suppression of epidemics and relief from disaster. The work of the so-called Moore Commission in New York State is an admirable application of this idea.

3. The states can refrain from passing laws which require expensive municipal services unless the state investigates to determine that municipalities can bear the burden or unless the state pays the cost of such services.

4. No state government at any time, anywhere in the United States, should have the right to fix the salary or hours of work of any municipal employee or official or any group of employees.

5. Many states can profitably examine their systems of grants-in-aid and shared taxes. In most cases, grants should replace shared taxes, but the grants should not be given solely to stimulate a particular type of expenditure, nor should the grants always be related to a particular expenditure. Grants should be given to increase local revenues and not to stimulate municipal expenditures.

6. The states should abandon the granting of special charters to municipalities but should govern through a municipal home rule law which gives specified responsibilities and privileges to every municipality and within specified limits permits each municipality to adopt and amend its own charters. Right now there is a good example before us. On Nov. 5, 1946, the city of Hartford, Conn., adopted a new charter by a three-to-one vote, favoring the city manager form of government. The charter was detailed. Now the legislature must ratify the charter. What a travesty on the democratic process it would be if the legislature should veto the charter! The cities should know what they can put in charters, and then they should be permitted to adopt them.

7. The gasoline tax should be more fairly distributed to the municipalities by the states. The greatest financial travesty of this century is the collection of the gasoline tax by the states, the use of the tax for rural roads and the construction of city streets by real estate taxes or special assessments.

8. For the protection of municipalities as public corporations and for the protection of municipal property owners, the states need to strengthen their laws on the use of land both in and adjacent to cities, so that the cities will not be surrounded by jerry-built houses on small plots of swampland, nor by types of buildings which the municipalities would not permit to be built. Municipal, county and state officials share the responsibility for protecting

present and future home owners from exploitation or depreciation.

9. A committee like the national "Baruch" committees with their simple, direct reports is needed to study and report on the areas of government, their number, their size and their ability to finance and perform services. How can a rural township in an urban area be expected to function in any way? It has no effective organization, no money, no technical or professional services. How can a city or village with one square mile of area properly function in a metropolitan region? Yet there are many of them. How can modern industrial city, particularly a one-industry city, be expected to care for its unemployed when nearly everyone is unemployed? Why should so many Illinois cities or townships have plenty of money for schools because there is a railroad track in the town, while neighboring cities or townships with equal or greater need cannot have good schools because an accident placed the railroad elsewhere? Why should the accidental or premeditated location of an industry give money and privilege to a small area, money and privilege to the particular industry, while denying services to equally worthy governments? Why should the accident of location give a government or industry privilege, while the accident of birth in the United States is supposed to confer special privilege on no one?

Sometimes these small areas of governments are knights that fight off the lords of the surrounding territory; more often they are vassals subject to the whim of economic chance, neglected by the state and existing in political poverty. At the other extreme, why should a great city like Chicago have to ask the state of Illinois for money? The city has the wealth and the income. Give it reasonable powers to tax and it will solve its own problems. To return to the question of areas. Examine the statutes, activities, revenues, officials and geographic boundaries of all the local governments in a state, and see if this question is not the most fundamental one any state can tackle.

Conclusions

How can these recommendations to the states and municipalities be carried out? Try this. Establish a state department of municipal affairs equal in rank to the department of highways, education or health. Give it authority to conduct research, gather statistics, prepare municipal laws and serve as the contact point between state and municipal officials, just as other departments do now in other fields. There is a need for the facts which can come only from uniform statistics. There is a need for the laws which can emerge from intimate departmental contacts.

Augment this department with legislative committees or interim commissions. The Pennsylvania local government commission has done notable work in eliminating a network of confusing legislation and substituting state-wide laws on debt and taxation. The Moore Commission in New York brought wide reform last year in the distribution of state aid and state grants. More recently the Maryland Commission on the Distribution of Tax Revenues has reported.

Finally, obtain the active co-operation of local government officials with the state department, state commissions and legislative committees. If all of these work together, such action may bring solutions to aggravating problems of tax reduction and financing of services.

If a dogmatic answer is wanted to the questions of whether taxes can be reduced and to what extent municipalities can support their public services, it would have to be that municipal taxes cannot be reduced substantially, but municipalities can support their normal traditional services if the areas of government are large enough and state revenues are distributed equitably. But a dogmatic answer is never the best, so a careful rearrangement of areas, better distribution of state taxes and a state agency for municipal affairs are recommended.

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Financing Improvements by Revenue or General Obligation Bonds

By William S. Evatt

A paper presented on Oct. 12, 1946, at the Ohio Section Meeting by William S. Evatt, Attorney at Law, Columbus, Ohio, formerly Ohio State Tax Comr. and Asst. Attorney General.

FINANCING public improvements by the issuance of revenue bonds, which are secured only by the revenue or property of an income-producing activity—or both—is of comparatively recent origin in this country. Traditionally, public improvements in America, whether or not they produce revenue, have been financed by the issuance and sale of general obligation bonds; that is, bonds secured by the taxing power of the issuing governmental authority.

The first public improvement financed by the issuance of revenue bonds in the United States was a water works issue in 1897 by the city of Spokane, Wash. In 1899 Illinois and Wisconsin, by statutory enactment, authorized this method of financing; and in 1909 Michigan adopted a revenue bond amendment to its constitution. Three years later the Ohio Constitutional Convention of 1912 took similar action which resulted in the adoption of Article XVIII, Section 12, of the Ohio Constitution.

Revenue Bonds in Ohio

The new section of the constitution expressly authorizes municipalities to issue mortgage bonds beyond the general limit of bonded indebtedness prescribed by law for the purpose of acquiring, constructing or extending any public utility. It further provides that such bonds shall impose no general liability upon the issuing municipality, "but shall be secured only upon the property and revenues of such public utility, including a franchise stating the terms upon which, in case of foreclosure, the purchaser may operate the same." Such franchises may not extend for a period longer than twenty years.

In other states there have recently been many revenue bonds issued by municipal corporations and other subdivisions for many types of revenue-producing public improvements, some of which are secured only by the revenue of the improvement for which they are issued. In Ohio, however, the only subdivisions of the state which may issue such bonds are municipalities, and even under the prescribed limits, the bonds probably may not be issued when secured only by the revenues of the utility. The constitution expressly provides that such bonds shall be secured by the property *and* revenues of the utility, including a 20-year franchise.

Notwithstanding the fact that there has been constitutional authority for municipalities to acquire, construct or extend municipally owned public utili-

ties by the issuance of mortgage revenue bonds since 1912, Ohio municipalities continued to finance such improvements by the issuance of general obligation bonds until the depression of 1929. At that time, shrinking tax revenues, sometimes accompanied by excessive municipal debt (chiefly in the field of special assessment bonds), the adoption of the ten-mill limitation, and the requirement of more than a majority on voted bond issues all caused municipalities to turn to mortgage revenue financing for their public utility improvements. A 60 per cent vote is required to carry a voted general obligation water works issue.

Heretofore, the chief function of bond attorneys consisted in passing upon the legality of general obligation bond issues. Usually they represented bond houses which purchased such issues and gave their opinions of the legality of all the proceedings leading up to the issuance and sale of the bonds. Since the advent of municipally owned public utility financing by the issuance of mortgage revenue bonds, the responsibilities of bond attorneys have been greatly increased. They are now required to prepare, consider and determine innumerable questions affecting the rights of the bondholders, the rights and duties of the trustee, and the rights and responsibilities of the city as set forth in the trust indenture. Many of these questions involve municipal finance and matters of business policy and practices as well as law.

In preparing a mortgage revenue bond issue for a public utility improvement, the interests of the municipality and of the municipal bond house, which buys the issue, are fundamentally parallel. Both bond houses and municipalities are interested in selling sound securities. The interests of the municipality cannot be served by issuing and selling a security which will result in default or foreclosure, thus jeopardizing its financial standing and reputation, and placing an indirect burden on the taxpayers who will have to pay higher interest rates on future financing, whether by general obligation or mortgage revenue bonds. The credit standing of municipalities is a vital factor taken into consideration by the bond house when bidding upon municipal bonds.

Investment Considerations

To decide whether it is advisable to issue general obligation of mortgage revenue bonds for a public utility improvement requires a determination of many factors. Generally speaking, general obligation bonds may be sold at a lesser interest rate than mortgage revenue bonds, but this is not always true. In Ohio in recent years, mortgage revenue bonds of municipalities have in many cases sold at as low an interest rate as would have been bid for general obligation bonds. There are several reasons for this. Some Ohio municipalities have been experiencing financial difficulties. General obligation bonds, therefore, have been thought to offer no better security than mortgage revenue bonds of a well-managed public utility in a good community. For example, the city of Toledo sold water works revenue bonds in 1939 at 2.73 per cent interest, and a few days later could secure no better bid than 3 per cent on general obligation bonds of similar maturity. Of course, as a general rule, revenue bonds issued for the acquisition of a new utility do not sell as favorably as such bonds issued to extend a successfully managed going utility.

General obligation bonds are eligible for purchase as investment by banks and for the investment of fiduciary funds, whereas mortgage revenue bonds are not so generally eligible, although moneys in the State Teachers' Retirement Fund, the Public Employees' Retirement Fund and other state trust funds may be invested in such securities. In a buyers' market, this has some effect on marketability.

Revenue bonds still enjoy the same immunity from federal taxation as general obligation bonds. There is concern in some quarters about the effect of the decision of the Ohio Supreme Court holding the property of the municipally owned Cleveland St. Railway, which was acquired by the issuance of mortgage revenue bonds, to be subject to state property taxation. It has been feared that this might stimulate a movement in Congress to tax all municipal revenue bonds. An effort has been made in recent years in Congress to tax all municipal bonds, general obligation as well as revenue, which so far has been defeated.

Advantage to Municipality

Probably the greatest advantage in issuing revenue bonds instead of general obligation bonds is that it adds flexibility to municipal revenue systems. Their issuance is not subject to either constitutional tax debt limitations or statutory debt limitations prescribing the aggregate of debt which may be incurred with or without the authority of the electors. The statutory and constitutional limitations of municipalities to borrow money for other necessary purposes are not thereby curtailed.

General obligation water works bonds are, however, less restricted by statutory debt limitations than are municipal power and light bonds. The Uniform Bond Act provides that water works bonds need not be considered in computing the 1 per cent unvoted nor the 5 per cent voted limitation of net indebtedness to the extent that they are self-supporting. A new issue, however, must be considered within these limitations, as an engineer's estimate that they will be self-supporting does not serve to take them out of the statute.

Another consideration which occasionally is a deterrent to issuing general obligation bonds is taxpayers' resistance to incurring new debt which might increase the tax burden. This objection is completely eliminated in revenue bond financing. Under such circumstances a self-supporting utility is accordingly enabled to carry on necessary expansion, and hence render more efficient public service, with a lesser degree of opposition.

A further result of sound revenue bond financing is that it sometimes makes possible more efficient administration of the utility. In order that revenue bonds may be sold most advantageously, the utility must be put on a sound business basis. The property must be adequately maintained, rates correctly adjusted and sound accounting methods installed. The investor in revenue bonds may look only to the utility, whereas the investor in general obligation bonds is protected by the taxing power.

Planning Bond Issues

In planning a revenue-bond-financed utility improvement, it is customary to enlist the services of competent, experienced consulting engineers. It is believed that no matter how experienced a municipal engineering staff may be, independent advice of a com-

petent outside consultant, if only for checking, serves to protect the municipal officials, the utility's customers and the investors, and thus aids in marketing the bonds.

After engineering aspects of the project are planned, legal services are required in planning the issue. The Ohio Supreme Court has held that the provisions of the Uniform Bond Act have no application to the issuance of mortgage revenue bonds under the constitution, and that the constitution is self-executing. This decision is often extremely helpful. The Uniform Bond Act requires that general obligation bonds mature serially in substantially equal installments and that, if maturing in annual installments, the first maturity may not be postponed more than approximately three years after the date of the passage of the bond ordinance; and, if maturing in semiannual installments, the first maturity must be not later than 2½ years after the date of passage. It is better practice to defer the first maturity date of a serial bond issue until a reasonable time after the improvement is completed and in operation—a period which may be more than three years. In these days of material shortages, the time factor assumes great importance.

The requirements of the Uniform Bond Act that bonds mature in substantially equal installments are perfectly sound in their application to tax-supported improvements. The theory is that the taxpayers should carry a greater burden during the first years while they have the benefit of, for example, a new schoolhouse, than they should carry fifteen or twenty years later, when the schoolhouse is old. But this reason does not hold for an income-producing utility which is self-supporting and financed from the proceeds of its own income. As long as proper provisions are made for replacements and reserves for depreciation, the security back of the bonds is not adversely affected when maturities are graduated in amount so as to place a more equal burden on the utility to service the bonds throughout the life of the issue. In fact, maturities so graduated may afford the investors a better security, particularly in the early years. If municipalities are bound by such provisions as those of the Uniform Bond Act in issuing mortgage revenue bonds, an inordinate burden is frequently placed upon the utility to service the issue during its earlier years.

Ordinance and Trust Indenture

There are many details which must be considered in adopting a revenue bond ordinance and trust indenture. A few of these should be mentioned.

For the protection of the municipality, as well as of the investor in mortgage revenue bonds, it should be provided that proceeds from the sale of the bonds, comprising the construction fund, be deposited in a bank or banks which are members of the Federal Deposit Insurance Corp., and be secured by a pledge to the city of government or surety bonds. The bond fund, consisting of revenues to meet the interest and principal requirements of the issue, should likewise be protected.

Since water works issues customarily extend over a period of approximately thirty years, it is impossible to foresee what the extension requirements of the municipality may be in the future, and, therefore, the issue should be what is called "open end" to allow for contingencies. Adequate protection, however, should be pro-

vided for the holders of the bonds being currently issued. The issuance of additional bonds should be authorized subject to the limitation that the earnings—after deduction for operation, maintenance, reasonable repair, replacements and depreciation charges—equal at least 120 per cent of the principal and interest requirements for the next ensuing fiscal year, not only of the present issue but also of the additional issue. Such interest and principal requirements of the additional issue should be determined by dividing the amount of such additional issue by the number of years to its final maturity date.

The trust indenture should contain ample requirements that the utility maintain insurance on all mortgaged property to its full insurable value. Such insurance should be satisfactory in form and amount to the trustee, who should be its beneficiary. The indenture should likewise contain provisions for the repair or replacement of damaged property and for the application of the proceeds of insurance.

In order that the municipality be unhampered in using sound business practices in operating the utility, provision should be made whereby the municipality may secure a release from the mortgage and sell property when it becomes no longer necessary or desirable for use in the operation of the utility. The municipality should also be permitted to dispose of chattels such as furniture, furnishings, equipment and machinery, free from the lien of the mortgage, when such chattels have become unserviceable and when such disposal is made for the purpose of replacement.

It is, of course, customary to require that the mortgagor keep complete and proper books of records and accounts on the operation of the utility, which should be available at all times to the trustee, and that the utility furnish to the trustee an income and expenditure statement semiannually, and a complete balance sheet annually.

It is also necessary that the trust indenture specify what constitutes an event of default and what the circumstances are under which the principal of all the bonds then outstanding may become due and payable immediately. It should contain full provisions on procedure and the rights of all parties in foreclosure.

There are, of course, many other details which must be considered in the preparation of a trust indenture to protect not only the investors and the trustee but the municipality as well.

Under the Uniform Bond Act, it is required that bonds which are not purchased by the sinking fund trustees may be sold only pursuant to competitive bidding after advertisement once a week for three consecutive weeks. There is no such requirement for selling mortgage revenue bonds. If the municipality desires to advertise such bonds and invite public competitive bidding, the proceedings may not be completed first, as with general obligation bonds. When such a course is followed, the trust indenture mortgaging the property, after having been tentatively agreed upon by the city and the trustee, may not be executed prior to the sale of the bonds, because the execution of the trust indenture, the execution of the bonds and payment to the municipality should take place simultaneously. It is customary in advertising such bonds to state that they will be secured by a trust indenture satisfactory to the purchaser.

Because of the basic differences between financing general obligation and

mortgage revenue bonds, municipalities frequently sell the latter at private sale, after having invited several reputable municipal bond houses to bid thereon. Another practice is to call in bond houses and enter into a contract for the bonds after the engineering work is sufficiently advanced and prior to the preparation of legal proceedings. On such occasions the municipality, the engineers and, frequently, legal counsel employed by the bond house work together in setting up an issue which will provide a sound financial program for the utility, adequate and fair rates for the consumers, protection for the municipality and an attractive issue for the investors which can therefore be sold at the lowest interest rate.

Conclusions

The future of revenue bond financing depends upon how this method of public borrowing is used. A report on this subject by the Haynes Foundation of Los Angeles stated (1):

> Turning again, by way of summary, to the significant general aspects of the subject—revenue bonds, if employed with discretion and with an intelligent regard to their limitations, can become a permanently important factor in public borrowing and a useful adjunct to the effective expansion of public services. They can readily, on the one hand, not only be discredited, but can be made to reflect discredit on the issuing public agencies. Designed to finance public enterprises operated on a self-supporting business basis, they throw the spotlight of publicity on the capacity of state and local governments to function with business-like efficiency. They possess the potentialities for increasing confidence in public institutions. To realize the potentialities fully, however, it will be necessary for legislatures to take constructive action in improving the statutes authorizing the use of revenue bonds, and for the users to apply them intelligently for legitimate purposes.

In view of the fact that the Ohio Constitution is self-executing, the question is not one for the legislature, but for the municipalities. As long as municipalities continue to work with and through legitimate municipal bond houses in this method of financing, the author has no fear that the broad powers conferred by the constitution will be abused, because legitimate established bond houses are just as interested in maintaining the high credit standing and integrity of the municipalities as are the municipal officials.

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Studies on False Confirmed Test Using B.G.B. and Comparison Studies on Lauryl Sulfate Tryptose Broth as a Presumptive Medium

By Miriam S. Shane

A contribution to the Journal by Miriam S. Shane, Bacteriologist, Water Dept., Wilmington, Del.

THE use of brilliant green bile lactose broth as a confirmatory medium has been widely recommended and used in water plant laboratories for a number of years. The U.S. Public Health Service Drinking Water Standards recognizes the use of this medium in the following words: "The confirmed test when the liquid confirmatory medium brilliant green bile lactose broth, 2 per cent, is used, providing the formation of gas in any amount in this medium during 48 hours of incubation at 37°C. is considered to constitute a positive confirmed test . . . (1)."

The Wilmington, Del., Water Dept. laboratory used the completed test for coliform organisms until the summer of 1943. After comparing the completed and confirmed tests using B.G.B. as a confirmatory medium and finding the results comparable, it was decided to use the completed test. When the temperature of the water in the fall and winter months became reduced, however, gas formation was evident in B.G.B. on 48-hour incubation, and the positive confirmed tests increased considerably.

Wilmington's plant is one of those that are inconvenienced by a great number of spurious presumptive tests, in which gas is formed in standard lactose broth in 48 hours. As early as 1931 an agreement between the Board of Water Commissioners of the city of Wilmington and Rutgers Univ. was arranged, and a scholarship was awarded to Lloyd R. Setter for the purpose of identifying the organisms responsible for the 48-hour gas formation in the department's presumptive tests. The work was carried on under the supervision of Prof. Willem Rudolfs. Through this investigation, a Gram-positive, sporulating organism was consistently found to be the cause of the spurious tests and was identified as *Clostridium multifermentans* (2).

The number of false positive presumptive tests seems to bear a direct relationship to temperature and rainfall. More false positive tests are obtained when the temperature of the water is low. The explanation is that the organism is spore-bearing and, when the temperature is low, it is in a spore stage and thus more resistant to chlorine. Other factors are also to be considered. Table 1 shows the relation of temperature to positive presumptive tests.

In view of the large number of spurious presumptive tests, and of the difficulty experienced with B.G.B., a

TABLE 1

Relationship of Positive Presumptive Tests to Water Temperature

Month	Avg. Temperature	No. of Samples Planted	Positive Presumptive Tubes	
	°F.		*no.*	*%*
July	80.2	1,055	223	21
August	79.3	1,065	257	24
September	72.0	1,005	187	19
October	59.0	1,035	213	20
November	48.6	990	213	22
December	36.5	1,030	402	39
January	34.0	1,035	370	36
February	35.5	950	436	45
March	49.2	1,045	204	19
April	60.5	1,000	101	10
May	62.9	1,060	206	19
June	73.3	1,015	242	24

TABLE 2

Comparison of Standard Lactose Broth and Lauryl Sulfate Tryptose Broth

Sample No.	No. of Tubes Planted	Positive Tubes in Lactose Broth		Positive Tubes in Lauryl Sulfate		Confirmed in Brilliant Green	
		24 hr.	*48 hr.*	*24 hr.*	*48 hr.*	Lauryl Sulfate	Lactose
1*	650	521	600	590	600	600	600
2*	650	391	522	524	560	545	501
3*	600	520	578	564	600	585	564
4*	800	260	630	232	269	267	264
5*	800	304	698	349	367	361	322
6	800	0	487	0	0	0	0
7	800	0	193	0	0	0	0
8	800	0	54	0	0	0	0
9	800	0	42	0	0	0	0
10	800	0	10	0	0	0	0
11	800	0	191	0	0	0	0
12	800	0	234	0	0	0	0

* Unfinished waters. Samples 1, 2 and 3 are raw, applied and basin waters, respectively.

two-fold experiment was attempted. Its purpose was, first, to determine the number of B.G.B. tubes that contained gas in 48 hours but not in 24 hours, and that showed the presence of the coliform group. The second objective was to make a comparison between standard lactose broth and lauryl sulfate tryptose broth in the cultivation of the coliform group and the inhibition of Gram-positive, sporulating, 48-hour gas-forming organisms.

Methods and Results

Material from each B.G.B. tube containing gas in 24 or 48 hours was

streaked on Endo's and E.M.B. media and incubated at 37°C. for 24 hours. Of 525 B.G.B. tubes containing gas in 48 hours, but not in 24 hours, not one showed any breakdown of either of the above media. It was also found that most transfers from a positive B.G.B. tube to a fresh tube did not grow; however, each time gas was formed in 24 hours, the coliform group was present, as evidenced by the breakdown of these media.

When material from the positive 48-hour B.G.B. tubes was stained, a large Gram-positive spore-bearing rod (usually in spore stage) and also a small Gram-positive rod were found.

Attempts to isolate these organisms in pure culture were successful with the large rod, but it did not produce gas in B.G.B. Attempts to isolate or cultivate the smaller organism have not been successful. Since the laboratory staff is not prepared to carry on anaerobic work, it only attempted isolation with Brewers' anaerobic medium.

Use was also made of 0.1 per cent agar in B.G.B. for second transfers from positive tubes, in order to establish anaerobiosis. The results were not consistent.

For the lauryl sulfate tryptose broth comparisons, the medium was prepared according to the formula used by Mallman (3), with the exception of the Duponol:

Ingredients

grams	
20.0	tryptose
5.0	lactose
2.78	dibasic potassium phosphate
2.78	monobasic potassium phosphate
5.0	sodium chloride
0.1	Duponol "C" *

* This product is a fine powder prepared by the Fine Chemicals Div. of the E. I. du Pont de Nemours & Co., Inc.

The above ingredients will make 1 liter of standard strength broth. Because of the dilution factor of the water samples, however, the amount of distilled water is reduced by one-third. The reaction is adjusted to pH 6.8.

The medium was tubed in 20-ml. portions and sterilized at 15 psi. pressure for 15 minutes. To bring the final concentration of broth up to the standard formula, 10-ml. samples were planted in 20-ml. portions of concentrated broth. All samples were planted in parallel in standard lactose broth and lauryl sulfate tryptose broth. Comparisons were made of raw water, applied water, filtered water before chlorination and water collected from regular sampling points in the distribution system.

All samples were planted in five 10-ml. portions, with the exception of the raw and applied waters, which were planted in five 1-ml. portions. All tubes showing gas in 24 or 48 hours were inoculated in B.G.B. and read at the end of 24 hours of incubation. The results are shown in Table 2.

Discussion

Studies over a two-year period show that 36 per cent of the B.G.B. tubes inoculated from positive presumptive tests exhibit gas formation in 48 hours with no evidence of the coliform group being present. The gas formation varies from a large bubble to 50 per cent of a Dunham fermentation vial. The majority of tubes, however, exhibited 10 per cent of gas or less.

The author is convinced that the organism responsible is a *Clostridium* which grows synergistically with an aerobic organism. The aerobic organism reduces the oxygen tension of the confirmatory medium.

In each instance when the coliform group of organisms has been shown to be present, gas was formed in the confirmatory medium in 24 hours. If gas is not formed in 24 hours, but forms in 48 hours and must be read as a positive confirmed test for the coliform group, it becomes necessary to report false positive results. As a result of these observations, the laboratory has been using both the confirmed and completed tests.

The results with lauryl sulfate tryptose broth bring out two facts:

1. Lauryl sulfate does not inhibit the growth of the coliform group when used in the concentration employed in this medium. In fact, a greater number was found when the lauryl sulfate tryptose broth was used.
2. Lauryl sulfate inhibits the growth of 48-hour Gram-positive gas formers.

One of the chief arguments against the use of lauryl sulfate was the fear of inhibiting the growth of weak strains of *Escherichia coli*. Because of the results obtained and the increase of positive confirmed tubes when lauryl sulfate tryptose broth was used as a presumptive test medium, this fear is considered to be unfounded.

Some laboratories advocate carrying enough residual chlorine in a distribution system to destroy all 48-hour gas formers. Water works people have the greatest respect for the efficacy of chlorine but are well aware that high dosages are objectionable to the consumer. It is considered a duty to the consumer to strike a practical balance between sterile and palatable water. The over-all answer to satisfactory plant operation and laboratory control is a city's freedom from water-borne epidemics.

As 48-hour gas-forming organisms are harmless and their spores resist relatively high chlorine dosages, the acceptance of lauryl sulfate as a medium in *Standard Methods* (4) is considered a great help in the reduction of time and media in daily routine technic in those plants inconvenienced by large numbers of 48-hour gas formers. Certainly it will solve the B.G.B. problems at Wilmington.

Acknowledgment

The author wishes to express her gratitude to Dr. Abel Wolman and Dr. M. H. McCrady for their interest and constructive suggestions. She is deeply indebted to W. Compton Wills, Mgr. and Chief Engr., and to the members of the Board of Water Commissioners of the Wilmington Water Dept. for their interest and co-operation in making this work possible.

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Colorimetry and Photometry in Water Analysis

By M. G. Mellon

A contribution to the Journal by M. G. Mellon, Prof. of Analytical Chemistry, Purdue Univ., Lafayette, Ind. Originally presented at the September 1946 meeting of the American Chemical Society, Chicago.

THE processing of water supplies to fit them for industrial use or human consumption is now so widespread that few think much about this very important chemical achievement. Closely allied is the treatment of sewage and industrial wastes to eliminate objectionable constituents or to reduce their effects to tolerable levels.

The control of the processing operations is a problem of analytical chemistry. The water works man must know the nature and amount of the significant constituents—usually before, during and after treatment.

Except for highly polluted samples, the amount of such constituents is nearly always relatively low. Even so, the concentration of some of them is of great importance. Some examples are: iron in laundry water, dissolved oxygen in boiler water and residual chlorine in drinking water. The emphasis is on the determination of minor constituents, as it is with certain other materials, such as steel and analytical grade reagents, in which the major constituent is not determined. As the desired constituents of water are usually in solution, the problem is simpler than it is with steel.

Handling minute amounts of constituents, some of which at times are present in amounts less than a part per million, presents real analytical difficulties. Some constituents, such as iron, can be concentrated to make a low sample more readily measurable; but this procedure is inapplicable for dissolved oxygen or residual chlorine. Perhaps gravimetry or titrimetry on a micro scale might be used, but such measurements for routine control are too exacting, both in equipment and technique.

One obvious approach to the analytical problem is to resort to methods designed for small amounts of constituents. Although colorimetric and photometric methods are not uniquely applicable in such situations, at present they seem for water analysis the best means available to determine various constituents. The present objective, therefore, is to survey briefly their contemporary status in this field.

In the author's opinion, the operations of any method of quantitative chemical analysis for most samples may be reduced to two processes: (1) rendering the desired constituent measurable, if it is not already so, and (2) measuring it. The first method involves chemistry; the second, physics. Since procedures used in water analysis are no exception, the subject is treated from this viewpoint.

Problems of Chemistry

As stated later, in the discussion of measurement, colorimetry is fundamentally the determination of the light-

absorptive capacity of the constituent in question. Following what seems generally accepted usage, photometry is a broader term, for it includes the absorptive capacity of a desired constituent for radiant energy in the ultraviolet, visual or infrared regions of the spectrum. To be susceptible to such measurement, the constituent must itself possess a suitable color, absorb selectively outside the visual region or react with some added reagent to produce such a quality. The general aspects of the chemistry in colorimetry have been considered in an earlier publication (1). In it, particular attention was devoted to the characteristics desired in systems destined for colorimetric measurement, and also to the requirements of reagents useful in developing usable colors.

For water analysis, the new edition of *Standard Methods* (2) lists one or more procedures for the following constituents: ions Al^{+3}, As^{+3}, BO_3^{-3}, Ca^{+2}, CN^-, CO_3^{-2}, Cl^-, Cr^{+3}, Cu^{+2}, F^-, Fe^{+2}, Fe^{+3}, H^+, HCO_3^-, I^-, K^+, Mg^{+2}, Mn^{+2}, Na^+, NO_2^-, NO_3^-, OH^-, Pb^{+2}, PO_4^{-3}, SiO_3^{-2}, SO_4^{-2} and Zn^{+2}; and dissolved gases Cl_2, CO_2, H_2S, NH_3 and O_2. Of these, not one is sufficiently colored, in the concentrations encountered, to be determined satisfactorily as such by a colorimetric method. It is possible, of course, that one might have a water containing such a colored constituent, as a permanganate. After treatment with reagents for developing suitable colors, colorimetric measurement is recommended for Al^{+3}, As^{+3}, Cl_2, CN^-, Cr^{+3}, Cu^{+2}, F^-, Fe^{+2}, Fe^{+3}, H^+, I^-, K^+, Mg^{+2}, Mn^{+2}, NH_3, NO_2^-, NO_3^-, Pb^{+2}, PO_4^{-3}, SiO_3^-, tannin and lignin.

To improve or extend such methods, three chemical possibilities are evident:

1. Better color-forming reagents might be found for present methods. Thus, after critical study of various reagents for developing colors with iron ions, several cyclic nitrogen compounds, such as 1,10-phenanthroline and 2,2′-bipyridine, were recommended (3, 4) in preference to the thiocyanate ion (5). Improvements in the control of present methods, such as those suggested recently for determining nitrates (6), are also possible.

2. Suitable color-forming reagents may be found for some of the constituents not now determined in this way. The great list of organic compounds, now nearing 400,000, has not been studied exhaustively for its analytical possibilities. There are two monographs containing much valuable information on selected lists of these compounds (7, 8). The revived interest in inorganic chemistry, a by-product of the uranium bomb, may yield something, for it seems unlikely that the chemistry of the compounds of all elements other than carbon has been exhausted.

3. Suitable reactants may be found for forming colorless complexes absorbing selectively in the ultraviolet or infrared regions.

Examples of studies of various chemical factors affecting certain colorimetric systems may be found in the papers by the author that are cited here (1, 3, 4, 5, 6, 9). Included is the procedure followed in establishing a recommended set of working conditions for a given method.

Problems of Physics

Once a system is ready for measurement, the chemical part of the determination is substantially, or completely done. The remaining part, measurement, comes within the scope of physics. The physical property of the desired constituent, or the system con-

taining it, that is selected for measurement depends upon various factors, including the nature of the constituent, its amount and the facilities available. The property will be either specific, such as its mass, or systemic, such as its color.

Color and Light

Strictly speaking, color is a psychophysical phenomenon—an experience of the observer, rather than a property of the object observed. What we really deal with, and measure, is the object's light-absorptive capacity. White light incident on the object is selectively absorbed, and the unabsorbed portion reaching the *macula lutea* on the retina of the human eye produces the sensation of color. The quality of the unabsorbed light is determined by the nature of the absorber, and the intensity by its amount. Although this discussion is limited to the visible ranges of radiant energy, or light, it seems not improbable that at least some of the constituents in water may be determined by measuring their absorptive capacity, or that of their compounds, in the ultraviolet or near infrared regions. Equipment is available, in an instrument such as the Beckman quartz spectrophotometer, with which such measurements are readily feasible between 200 and 400 mμ, and between 700 and 1,100 mμ, in addition to the visible region of 400 to 700 mμ.

The basic elements of a combination designed for colorimetric measurement are the illuminant, usually a near-white light, the colored solution of the desired constituent and the receptor for the unabsorbed light. A general outline of the principles and possibilities of such equipment has been presented elsewhere (9, 10). The present discussion is limited to a brief statement on what now seem to be the most useful instruments for water analysis. They are considered from the viewpoint of the means employed to determine the light-absorptive capacity of the unknown. All such measurements depend, of course, upon a knowledge of the relationship between this capacity and the amount of the desired constituent.

Color Comparometers

In the simplest type of color comparometer, or comparator, the light-absorptive capacity of the unknown solution is determined by comparison with that of a solution of known concentration, or with something colorimetrically equivalent to it. There are four techniques for doing this: the standard series, balancing, dilution and duplication. The first two of these are of chief interest to water analysts.

The standard series technique has long been familiar to users of *Standard Methods* (2). It consists of comparing the unknown solution with a series of knowns, the value of the color-matching known being taken as a measure of the concentration of desired constituent in the unknown. Nessler tubes are very useful for this purpose, the form 30 cm. in length enabling a determination of surprisingly small amounts of constituents such as ammonia or residual chlorine. The sensitivity is high with this method.

In order to have the standards simulate the unknown as closely as possible optically, it is often advised that they be prepared in the same manner, and contain the same materials, as the unknown. If time is a factor influencing their colorimetric value, the solutions may have to be prepared along

with that of the unknown. Preparative difficulties with certain systems, such as that for residual chlorine, make this type of standard practically unusable. The alternative has been a variety of "permanent" standards, the most used of which are solutions and glasses. Solutions especially must be watched for inconsistency in the series. Occasionally individual members fade or change hue. Then there is the question of spectral similarity of the transmitted light for the standards and the unknown. Very few different systems have identical spectral transmittancy curves, though the solutions may appear visually matched. Different curves for the unknown and the standards can mean difficulty in matching unless the spectral quality of the measuring illuminant is the same as that used in establishing the standards. Since the unknown and known solutions in a set of standards have the same depth, possible nonconformity of the system to Beer's law is immaterial.

Series of glass disc standards are available for a number of commonly determined constituents. Such equipment is probably reasonably permanent, and generally somewhat more convenient than solution standards. The difference in optical properties of the solid and liquid systems is probably not serious for any ordinary color comparison. Presumably nonconformity of the unknown solution to Beer's law must be provided for in the standardization of the glasses.

With the balancing, or Duboscq, type of comparometer, the system must either conform to Beer's law, or a correction be made for significant deviations. This technique assumes that the concentration of the solution is inversely proportional to the depth measured. Micro cups for such instruments provide for working with small volumes of solution.

Probably the chief merits of comparometers are their simplicity and economy. Properly handled, on systems for which they are applicable, they will yield reliable data of sufficient accuracy for the usual water analysis, although they are fatiguing if used for long periods. They are also generally inapplicable for multicomponent colored systems, such as those encountered in the determination of chromium and manganese in steel, or of the two chlorophylls in green leaves.

Absorptometers

In contrast to comparometers, color absorptometers are designed to determine the actual light-absorptive capacity of the system measured. They also differ from most comparometers in having means for limiting the incident light of the illuminant to regions of the spectrum much narrower than 400-700 mμ, the visual range. As this general subject has been covered elsewhere (9) more fully than is warranted here, the present considerations are limited to the problem of applying such equipment in water analysis.

Instruments

In order to understand the application of absorptometers, certain well-known facts will be reviewed. These concern the illuminant, the colored solution and the receptor, which is the human eye in visual instruments, and a photocell in photoelectric instruments.

Although sunlight might serve as the illuminant in certain instruments, a more readily controllable experimental source is an incandescent electric light bulb. The relative spectral distribution of the radiant energy

emitted at different wave lengths by such a source at 3,000° K. is shown by Curve *A* in Fig. 1. If only the two bands, 400–420 mμ and 680–700 mμ, were allowed to pass, it is obvious that their intensities would be quite different. Such a limited range of wave lengths is known as the spectral band width. Occasionally an illuminant may be limited to a single spectral line by use of a monochromatic source, such as the mercury arc line at 546 mμ.

Turning now to the two common receptors, the human eye and photocells, we find that their response char-

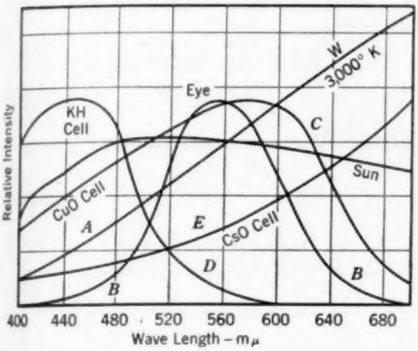

FIG. 1. Relative Intensity of Energy Emission and Response

acteristics vary widely for different wave lengths of radiant energy, or colors, as shown in Fig. 1. Curve *B* shows that the normal human eye is most sensitive to wave lengths just above 550 mμ, and that it is relatively insensitive below 430 or above 680 mμ. This curve is displaced at least 10 mμ to the left for the dark-adapted eye. Users of visual instruments must beware of high percentage errors when the radiant energy of the illuminant is restricted to one of these regions. The use of such instruments may also involve a serious fatigue factor in shifting back and forth from the dark-adapted state, sometimes required for photometric matching, to the light-adapted state for reading instrument scales.

Photocells of different types, or even of the same type, have a variety of response characteristics depending upon the materials and methods of production. Curves *C, D* and *E* of Fig. 1 roughly illustrate three types. Curve *C* is for the photovoltaic or photronic type. Its relative sensitivity for different wave lengths roughly parallels that of the eye above 550 mμ, but below this value the cell is better. Even so, with this type of cell, one can not expect the same instrumental response in the far blue as in the yellow-green. Most of the common photoelectric filter photometers use such cells.

The photoemission type of cell has different response peaks, depending upon the cathode coating. Curve *D* of Fig. 1 is for a potassium hydride cell, and Curve *E* for a cesium oxide cell. It is evident that an instrument having high sensitivity over a wide range of wave lengths would require more than one cell. This necessity is recognized in the Beckman quartz spectrophotometer, which is equipped with two cells of quite different, but overlapping, ranges, so that one may use whichever is most appropriate for the system to be measured.

It is beyond the scope of this paper to consider the merits of the design and operating characteristics of photocells in absorptometers. They have been incorporated in instruments in a variety of ways, and usually the incorporators are ready to defend their products. Suffice it here to state that no one instrument seems to possess all the merits, and that they are not all equally good for all purposes.

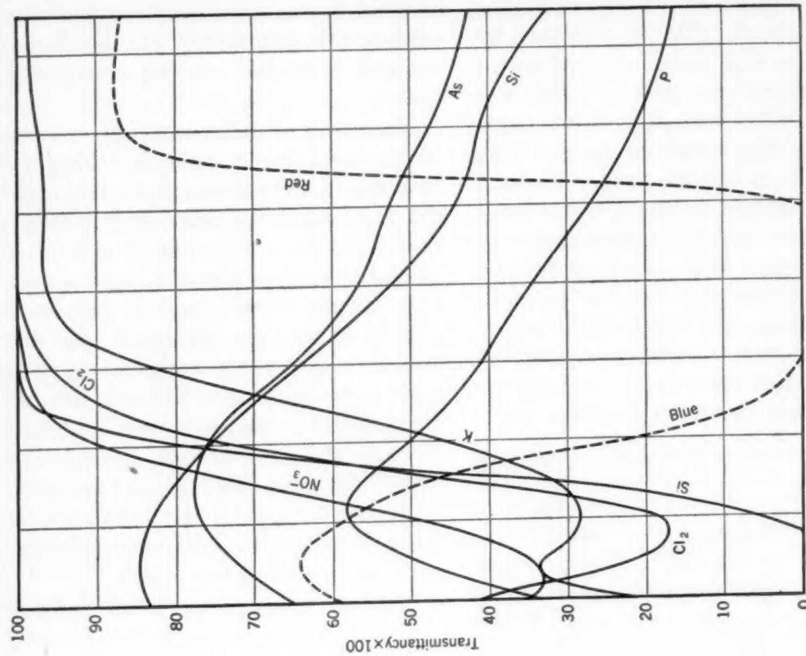

FIG. 3. Spectral Curves Showing Transmittances of Blue and Red Glass Filters and Transmittancies of Yellow and Blue Solutions

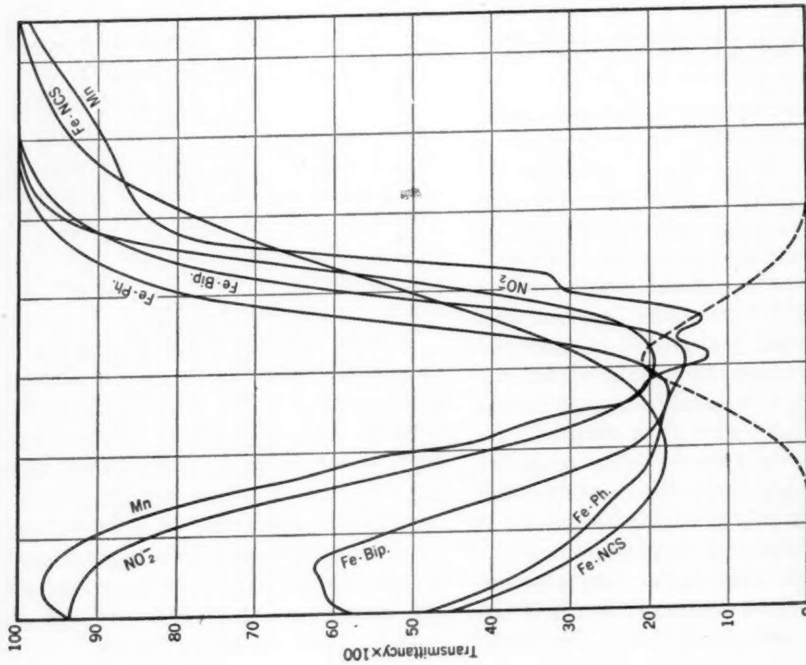

FIG. 2. Spectral Curves Showing Transmittance of Green Glass Filter and Transmittancies of Reddish Purple Solutions

Before considering the appropriate use of such instruments, the light-absorptive characteristics of the solutions must be considered. For water analysis the systems now recommended for colorimetric measurement may well be taken as examples. Figures 2 and 3 show the spectral transmittancy curves for part of these solutions. The percentage of light transmitted is plotted as the ordinate in each figure, against the wave length as the abscissa. The data were obtained with either a General Electric recording, or a Beckman non-recording, spectrophotometer, operating on the band widths and with the absorption cells shown in Table 1. The systems range in hue from yellow through reddish purple to blue. In Fig. 2, the broken line represents the transmittance of a green glass filter. The solid lines represent the transmittancies of various reddish-purple solutions: Mn as MnO_4^-, NO_2^- with 4-aminobenzenesulfonic acid and 1-aminonaphthalene, Fe^{+2} with 1,10-phenanthroline and 2,2′-bipyridine, and Fe^{+3} with thiocyanate. In Fig. 3, the broken lines represent the transmittances of a blue and a red glass filter, and the solid lines the transmittancies of the following solutions: (yellows) Si as

TABLE 1

Band Widths and Absorption Cells for Fig. 2 and 3

Constituent	Reagent	Concn. *mg./l.*	Cell Length *cm.*	Band Width *mμ*
As^{+3}	Ammonium molybdate Hydrazine sulfate	3.0	1	10
Cl_2	*o*-Tolidine	0.2	5	5
Fe^{+2}	2,2′-Bipyridine	5.0	1	10
	1,10-Phenanthroline	4.0	1	10
Fe^{+3}	Ammonium thiocyanate	6.0	1	10
K^+	Sodium cobaltinitrite Potassium dichromate	10.0	1	1
Mn^{+2}	Potassium periodate	10.0	2	5
NO_2^-	4-Aminobenzenesulfonic acid and 1-aminonaphthalene	0.4	2	5
NO_3^-	Phenol-2,4-disulfonic acid	1.0(N)	1	1
PO_4^{-3}	Ammonium molybdate 1-Amino-2-naphthol-4-sulfonic acid	6.0(P)	1	10
SiO_3^{-2}	Ammonium molybdate 1-Amino-2-naphthol-4-sulfonic acid	3.0(SiO_2)	1	10
SiO_3^{-2}	Ammonium molybdate	10.0(SiO_2)	20	10

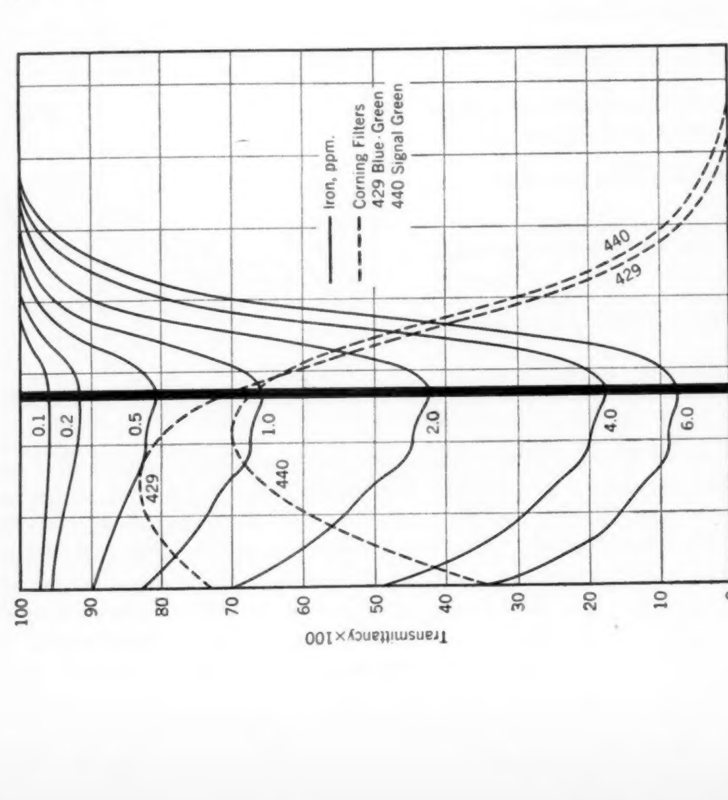

FIG. 4. Spectral Curves Showing Transmittances of Two Green Glass Filters and Transmittancies of Various Iron Solutions

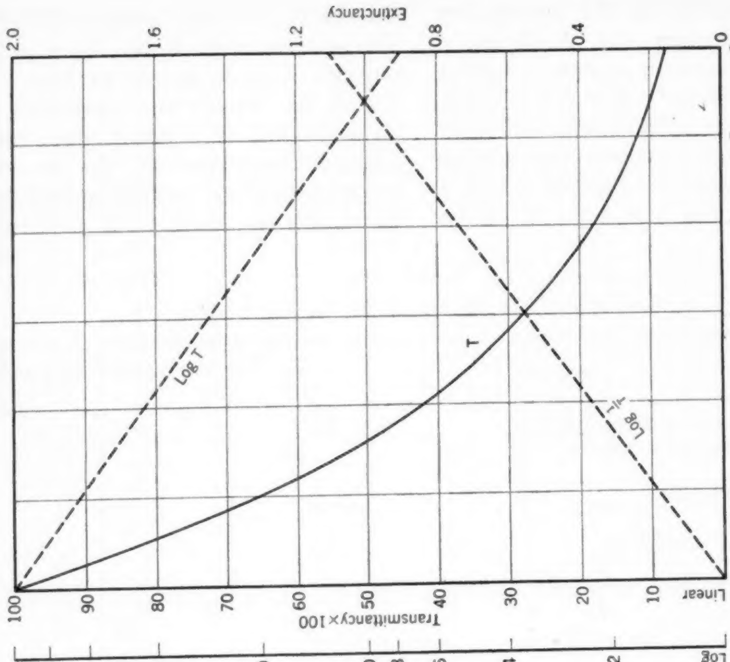

FIG. 5. Calibration Curves for Iron Solutions of FIG. 4

$H_4[SiMo_3O_{10})_4]$, Cl_2 with ortho-tolidine, K through reduced dichromate and NO_3^- with phenol-2,4-disulfonic acid; (blues) Si, As and P as heteropoly blues.

As a general principle, it is desirable in quantitative work to measure the light-absorptive capacity at the wave length of maximum absorptancy of the solution. In this region there is the greatest difference in transmittancy between any two different concentrations, as shown in Fig. 4. This means maximum sensitivity, assuming instrumental response for the region selected. With this in mind, the instrument is set to pass a light beam of the desired wave band. The curves in Fig. 2 show minima, but those in Fig. 3 do not, except for chlorine, nitrate and potassium. If there is no minimum, the only possibility is to work in the best region available. Of course, an instrument having the versatility of the Beckman quartz spectrophotometer enables one to move into the ultraviolet or near infrared, if desired.

Broadly speaking, the desired spectral band may be isolated by means of filters or monochromators, giving rise, respectively, to filter photometers and spectrophotometers. This isolation involves two considerations—the width of the spectral band and its location on the wave length scale. Examples are given in Fig. 4. The solid line curves are for a series of solutions of iron, ranging in concentration from 0.1 to 6.0 ppm., using the color-forming reagent 1,10-phenanthroline. The peak of the absorption band is close to 508 mμ. The broken lines are for two Corning glass filters, having maximum transmittances near 460 and 485 mμ. The higher value comes nearer coinciding with the minimum value for the iron solutions; hence this would be the preferred filter. It should be noted, of course, that the spectral band passed by the filter is wide; but most of the light is excluded in the region above 550 mμ, where the high transmittancies for the iron solutions occur. By using multiple filters, the spectral band may be narrowed considerably, usually with much loss in sensitivity, unless there is provision to compensate this effect in the electrical design. Some instruments have 8 to 10 filters, but one type has only three. The curves for these three are shown in Fig. 2 and 3. The green one serves for the reddish purples in Fig. 2, and the red and blue ones in Fig. 3 for the blue and yellow solutions, respectively. Preferably the selection of a suitable filter presupposes a knowledge of the spectral transmittancy curve of the solution, as well as the transmittance of the filter.

The wide line in Fig. 4, centering on 508 mμ, represents the band passed by a spectrophotometer set for 5 mμ. The versatility of the better spectrophotometers lies in the means provided for using spectral bands of desired widths and for locating them at any desired wave length. Such equipment varies widely in quality in this respect, one commercial instrument being designed to operate only on 35 mμ. In contrast, at least one non-commercial instrument will operate as low as a few Ångstrom units.

This brief presentation concerns thus far only single component, colored systems. Space can not be taken for discussing multicomponent systems, nor has water analysis been concerned much with them. In general, they are not measurable with filter photometers, although Knudson, Meloche and Juday (11) worked with one such system. The highest grade spectrophotometers find their best use for such

purposes, examples of which have been described elsewhere (9). It seems possible that such an application might be feasible in water analysis; if so, two or more constituents would be determinable in the same sample.

Absorptometric Data

Presumably any colored solution may be measured absorptometrically if the colored component is sufficiently stable. One must know, of course, that the absorption is caused by the desired constituent. Only the general problem of dealing with the relevant data will be reviewed here.

Once the filter photometer or spectrophotometer appropriate for the system to be measured has been selected, the light-absorptive capacity of the desired constituent in known amount must be determined. That is, the instrument must be calibrated under the given working conditions, such factors being involved as the filter or monochromator setting, the absorption cell and any interfering constituents necessitating compensation or correction.

Although there are various ways of using absorptometric data (9), only the two most common methods will be mentioned. In the first, a calibration curve is constructed relating the instrumental reading, as the ordinate, to the concentration, as the abscissa. In Fig. 5 three possibilities are illustrated for the iron solutions of Fig. 4. The solid line curve is for transmittancy, as taken from a direct-reading spectrophotometer. The broken lines are for logarithmic values. The one passing through 100 is plotted on the logarithmic ordinate scale, but plotting the logarithm of the transmittancy on a linear scale would give the same result. The one passing through zero uses $\log \frac{1}{T}$. Some instruments read directly in the latter, or extinctancy, values. However the values are plotted, construction of such a calibration curve involves determining them for a series of solutions of known concentration of the desired constituent under the conditions to be used later. Because of the effect of possible instrumental factors, each photometer should be calibrated for its intended use.

If extensive qualitative use is to be made of the curves for identification of constituents, the data should be plotted with the logarithm of the extinctancy as the ordinate, since the curve form is then the same for all measurable concentrations (12).

It may be mentioned that the linearity exhibited by the broken line curves of Fig. 5 shows conformity of the system to Beer's law. For such systems the Bouguer-Beer relationship:

$$c = \frac{\log \frac{1}{T}}{E_m b}$$

may be applied without a calibration curve, c being the concentration of desired constituent in mols per liter, T the transmittancy, b the cell thickness, and E_m the molecular extinctancy for the given cell (generally defined for 1.000 cm.). The value of $\frac{1}{T}$ is determined experimentally, b is known, and E_m must be determined for a solution of known concentration. As mentioned above, careful work requires checking the value of E_m for the given instrument, even though others may have reported such data.

Applying this procedure to the determination of iron (Fig. 4, curve for 4 ppm.), the transmittancy is 0.18 (18 per cent) at wave length 508 mμ

Since the concentration may be taken as 4 mg. per l. (0.0000716 mol) and the cell thickness is 1.00 cm., the constant E_m is 10,400 for the curve determined at 10 mμ band width. This value may then be used for the system, under the conditions specified.

Summary

Because of their small amount, their nature, and the condition in which they are found, many of the constituents in waters may be determined advantageously by colorimetric methods. Water analysts have been alert to this possibility and have approved such methods for more than half of the constituents included in the new edition of *Standard Methods*. Extension of this number seems possible.

Present instruments seem to be adequate for the necessary measurements. Comparometers are simple, economical and generally satisfactory for visual determination of single-component colored systems. Photometers may be substituted in practically all such work. In addition, by providing greater selectivity, they may enable one to avoid interfering colors. The best spectrophotometers are adaptable to determining various multicomponent colored systems. All such photometers, when equipped with photoelectric receptors, eliminate certain subjective difficulties inherent in visual instruments.

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Stream Pollution Control in Tennessee

By R. P. Farrell

A paper presented on Oct. 30, 1946, at the Kentucky-Tennessee Section Meeting, Knoxville, Tenn., by R. P. Farrell, Director, Div. of San. Eng., State Dept. of Public Health, Nashville, Tenn.

THE progress in stream pollution control in Tennessee parallels very closely the history and development of the Division of Sanitary Engineering of the Tennessee Dept. of Public Health, and, of course, is closely related to other public health activities. During the period from 1925 to 1936 the stream pollution problem, particularly in its relation to public water supplies and sewage disposal, was studied, and an active program inaugurated. At the same time as this program was continued, studies were made of laws in other states, with a view toward the submission of a proposed law on stream pollution in Tennessee. The division also participated actively in the Ohio River Interstate Agreement. The work done was of value in unifying the thinking in the several states involved. After considerable study of legislation in other states, an act, somewhat similar to that of Wisconsin, was submitted to the Tennessee Legislature in the early 1930's. Although this act was not passed, an engineer was employed to make special studies of stream pollution problems in Tennessee.

History of Control Efforts

From 1936 to 1942, special studies were made of the Cumberland River by the Division of Sanitary Engineering of the State Health Dept., and intensive studies were made of the Tennessee River Basin by the Tennessee Valley Authority. Similar data were obtained by both agencies, the procedures based on work previously done, and being done, by the U.S. Public Health Service Experiment Station at Cincinnati, Ohio. Also, a study of pollution in the Ohio River Basin was made by the U.S. Public Health Service. This study included both the Cumberland River and the Tennessee River Basin, and supplemented the work which had already been done by the Tennessee Valley Authority and the state. Another bill was submitted to the State Legislature for the control of stream pollution; but, due to the pressure of other matters, this bill was never introduced.

Although no active work was done—beyond that involved in routine water and sewage programs—during the period 1942–1945, active studies were continued by the TVA, and it appeared wise to sponsor legislation again in 1943. The bill still was not enacted, but the legislature authorized the Governor to appoint a Study Board to report conditions of pollution in Tennessee to the next session of the legislature. Fortunately, there were sufficient data in the division files, including the material from the TVA, U.S. Public

Health Service, and that collected by the State Health Dept., to assemble a report showing the extent of pollution in Tennessee. A Summary Report was completed during the latter part of 1944 and printed in time for submission to the Governor and members of the legislature early in 1945.

Data assembled in the report showed that there were about 15,000 miles of streams in the state, that approximately 710 miles were seriously polluted and that 850 additional miles were unsuitable for many desirable uses. It was noted that sewage from about 936,000 persons was being discharged into Tennessee streams, and that the pollutional load from domestic sewage was reduced only 7.4 per cent. It was also found that there were 280 industrial plants in the state contributing wastes equivalent to the sewage from 1,267,000 people, and that the pollution load from these wastes was reduced less than 1.0 per cent by treatment. Apparently, these data were sufficient to indicate that some means of control was necessary, for the present law was enacted early in 1945.

The act, as passed, created the Tennessee Stream Pollution Control Board consisting of the Commissioners of the Departments of Health, Conservation and Agriculture, and four appointed members, two representing industries and two representing municipalities. This board was given the authority to regulate and control pollution of surface waters of Tennessee. To carry out the intent of the act, the board was authorized to make investigations, approve plans, issue permits, establish standards, adopt regulations and issue orders for correction of pollution problems. Also, penalties for violations of the terms of the act were included in the law.

Activities of the Board

The board was appointed by the Governor and held its first meeting on May 25, 1945. Since the law designated the Commissioner of the Dept. of Health as Chairman, the Director of the Division of Sanitary Engineering as Technical Secretary, and the Commissioners of the Conservation and Agriculture Departments as members, the only organizational formality necessary at the time of the first meeting was the taking of the oath of office by the appointed members. The board was ready to transact business immediately, and started in on a discussion of policies, procedures and general regulations. A tentative draft of General Regulations was prepared at the time of this first meeting. This was later reviewed by a representative of the Attorney General and adopted as official.

The General Regulations established procedures to be followed in the:

1. *Supervision of new construction.* Control is effected by requiring the submission of preliminary plans; samples, as necessary; complete working plans; and records of existing works, where necessary to determine the extent of pollution.

2. *Supervision over maintenance and operation.* Controls require the keeping of records and reports of operation; routine submission of samples; and reliable operation of treatment facilities. Evidence of competency may be required if deemed necessary to insure proper operation of the disposal works.

3. *Investigations, recommendations, criteria and orders.* There is outlined a policy of investigation of complaints or reference to the proper local agency for investigation and action. Also covered is the policy of special

investigation of existing or new disposal works, to be followed by reports and recommendations regarding needed improvements. Standards or other criteria will be also prepared and approved by the board as a guide to the Dept. of Health in the administration of the Stream Pollution Control Program. The board may issue general or special orders relating to pollution.

4. *Permits to discharge sewage, industrial wastes or other wastes.* This section outlines the procedure for filing applications for permits to discharge sewage or industrial wastes, and provides for the issuance of two types of permits and the revocation of permits for cause.

After the adoption of these General Regulations, copies were mailed to all municipalities and industries in the state. The board further decided that the plan of action should include special study of the problem by river basin areas. Each basin will be studied, starting at the headwaters and working downstream, in order that the work of each municipality and industry may be co-ordinated with the activities of the Stream Pollution Control Board. Only in this way will it be possible to solve the pollution problem in any particular section or stream.

Although no funds were provided by the legislature specifically for stream pollution control work, the board approved the employment of personnel within the limits of funds which could be made available by the Departments of Health, Conservation and Agriculture. This staff includes a principal sanitary engineer, a part-time consultant, a chemist and a stenographer. It is planned to employ additional engineers, chemists or chemical engineers, bacteriologists, biologists and the necessary assistants and clerical help to carry on a full-time program of field study, together with laboratory work and administrative details in the central office.

A sanitary chemical laboratory in Nashville is being developed more fully for stream pollution control work, and two army surplus laboratory trucks have been purchased and are to be equipped for chemical, bacteriological and biological field studies. A station wagon has also been ordered to service field laboratories. Equipped with a minimum amount of laboratory equipment and reagents, it can be used in the investigation of complaints.

The board authorized the appointment of a Technical Committee to include representatives of the State Planning Commission, Division of Game and Fish, the State Dept. of Agriculture, a municipal water department, the TVA stream sanitation staff and two representatives of industries. This committee has been actively at work in the preparation of "General Criteria for the Definition and Control of Pollution in the Waters of Tennessee." A preliminary report has been prepared by this committee in co-operation with members of the technical staff of the Tennessee Stream Pollution Control Board, and has been submitted to various federal agencies for review and comment prior to its submission to the control board. The report of the Technical Committee includes minimum criteria of water conditions in terms of solids, temperature, turbidity or color, taste or odor, acidity or alkalinity, hardness or mineral compounds, dissolved oxygen, toxic substances and other pollutants.

Policies and Procedures

The board has approved a co-operative program in which the state as-

sumes full responsibility for the study of streams, and municipalities and industries assume responsibility for sewage and wastes, respectively. It is assumed that the state will not only make studies necessary to determine the amount of pollution in streams and the degree of correction necessary by various municipalities and industries, but will also guide both municipalities and industries and outline to them the studies which will be necessary to serve as a basis for corrective works. In addition to its responsibility to acquaint industrial and municipal officials with the program of the Stream Pollution Control Board, the Division of Sanitary Engineering has outlined and is recommending three steps in the solution of each particular problem: to make a study of the sewage or waste problem to determine the extent of correction needed, if any; to prepare detailed plans and specifications for treatment as indicated by the study, together with studies of the stream by the state; and to construct the needed treatment facilities.

Co-operative Agencies

As previously mentioned, the TVA has been carrying on an active stream pollution study program. The division's control program has been integrated very closely with that of the TVA, as it is desirable to use all existing resources in order to expedite field work. The board is indebted to the TVA for making its field laboratory facilities and personnel available for special study work in the valley area. Data obtained by the TVA laboratory, in co-operation with members of the staff of the Stream Pollution Control Board, will be made available to the board for interpretation and use in the control program. In addition to the assistance in the study of streams, the TVA has agreed to make data available which are already in its files, and is also co-operating with municipalities and industries in the study of special problems.

Work has already been started in the Holston River Basin, where pollution stems from the cities of Elizabethton, Johnson City, Bristol and Kingsport, as well as several major industries. The data obtained on municipal sewage or industrial wastes will be made available to the industries or municipalities through the staff for their use in designing necessary treatment works. Normally, it would not be thought possible to make detailed studies of sewage and industrial wastes, as that is the responsibility of the municipality or industry. It is being done in the Valley Area only because of the willingness of TVA to participate in this type of work as an aid in establishing the program.

The Dept. of Conservation has co-operated to the extent of furnishing a biologist to work with the staff on special studies of stream pollution, particularly effects upon fish life.

The State Planning Commission has assisted by making special studies of communities and their needs, particularly as such needs may relate to the development of sewage systems.

There have also been conferences between representatives of the states of Tennessee and North Carolina on interstate pollution problems. Both of these states are actively working on their stream pollution problems and will co-ordinate their efforts on interstate problems.

Except for the TVA very little assistance has been received from federal agencies. The U.S. Public Health Service has offered to give consultant

service, however, and one visit has already been made by a representative of the service to study problems in the Holston River Basin. Further services are anticipated in the future.

Progress and Accomplishments

Although progress on stream pollution control appears to be a little slow, the establishment of the mechanism for a control program is quite involved. The board's regulations require all municipalities and industries discharging polluting materials to any surface stream of Tennessee to file an application to continue such discharge, and also provide that any new industry must obtain a permit before wastes may be discharged. Special forms on which applications were to be made were prepared and sent out. To date 352 applications have been received for permission to continue to discharge wastes into Tennessee streams. Only 25 permits have been issued; these are tolerance permits allowing time for further study of the problem and the design of needed treatment works. Additional permits will be issued as rapidly as field visits can be made to determine the conditions under which permits may be issued.

Since the special studies started in the Upper Holston River Basin, all the municipalities and industries in that area have started special studies or have employed engineers to make studies and prepare plans for treatment works. In all, 41 municipalities have begun to study their problems and to plan sewage treatment facilities and needed improvements to their sewer systems. Of these communities, 22 have received aid for engineering planning from the Federal Works Administration.

It is believed that as many projects are now under study as would be feasible at one time. The division hopes to encourage municipalities and industries to make studies throughout the state as rapidly as engineers are available for such work. Although no project is yet under construction, many will be constructed as soon as the material and labor situation is more settled.

Effect of Density Currents Upon Raw Water Quality

By M. A. Churchill

A paper presented on Oct. 30, 1946, at the Kentucky-Tennessee Section Meeting, Knoxville, Ky., by M. A. Churchill, Hydraulic Engr., Hydraulic Data Div., Tennessee Valley Authority, Knoxville, Tenn.

DENSITY currents exist and cause some unique effects in the arm of Watts Bar Reservoir that furnishes the public water supply for Harriman, Tenn. The Harriman water plant intake is located approximately a mile below the extreme upper limit of backwater on the Emory River arm of the pool, and about thirteen miles above the junction of the Emory and Clinch River arms of the pool. Past experiences have indicated that, under the existing backwater conditions, cold water releases at Norris Dam into the Clinch River can run upstream in the warmer water of the Emory. Such upstream flows materially affect the quality of the raw water drawn into the Harriman water plant.

At the Harriman water plant location, the depth of pooled water is about 25 ft. During the winter months, or whenever fairly high flows from the Emory River headwaters exist, the direction of the stream flow in the entire cross-section of the pool is downstream past the water works. During the summer months, however, when low flows normally exist, density currents have been observed throughout the entire 14-mile length of the Emory arm of the pool.

According to one definition, "a density current is a gravity flow of a fluid through, under or over, a fluid of approximately equal (but slightly different) density." A common example of a density current is the flow of cold air across the floor of a warm room when an outside door of a house is opened in the winter.

The effect of the junction of the Emory River arm of the reservoir—relatively warm during the summer—and the Clinch River—relatively cold and therefore slightly heavier—is to cause upstream density underflows of cold Clinch River water in the Emory arm of the pool. Clinch River water is colder than that in the Emory because cold water from the bottom of Norris Reservoir is released into the Clinch River 76 miles upstream from its junction with the Emory. The water at the reservoir bottom is cold during the summer months because, having cooled off during the previous winter and having settled to the bottom of the reservoir, it does not have the same opportunity of warming up as summer approaches as does the water flowing down from the headwaters of the Emory River.

Backflow of Wastes

As the cold Clinch River water flows up the Emory arm of the pool along the bottom as a density current, it

flows past the Harriman sewer outlets and also past the outfall from a large paper mill. Sewage and mill waste are discharged into the cold water current and are carried by it upstream to the intake of the Harriman water plant, located about one and one-half miles above the paper mill outfall.

The upstream density underflow is maintained during the summer months so long as the flow of the cold Clinch

bayment. Clinch River water is so much higher in alkalinity that Emory River water that it can be located very accurately at any time by analyzing a series of samples collected both longitudinally and vertically throughout the length of the Emory River arm of the pool.

The upstream movement of Clinch River water into the Emory embayment along the bottom displaces an

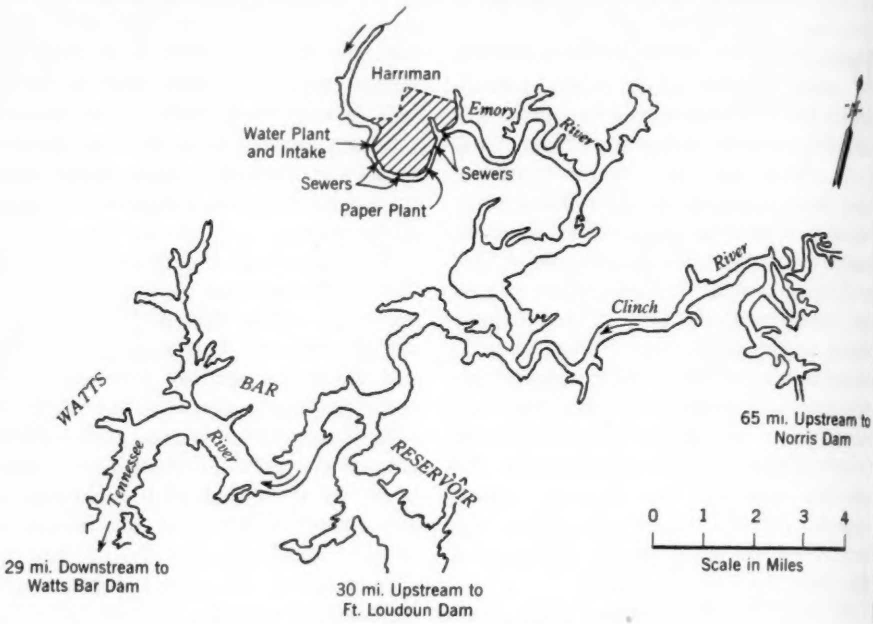


FIG. 1. Map of Watts Bar Reservoir and Vicinity

River water past the mouth of the Emory River arm is maintained and the inflow from the Emory headwaters remains low. Should Norris Reservoir be shut off, however, the cold water which is in the Emory River arm will drain back out of the Emory arm along the bottom. The velocity of upstream movement is variable, but movements as fast as one mile a day have been measured by titrations of water samples collected from the em-

equal quantity of water which must flow out of the Emory arm into the Clinch River arm of the pool near the water surface. In this way, a longitudinal circulation is set up which extends throughout the entire length of the Emory River arm of the pool.

The Harriman sewer outfalls are located near the bottom of the river. The original outfall from the paper mill is also near the bottom of the river. It was apparent soon after the

beginning of the studies described that it would be advantageous to build a new outfall at the mill so that the mill's "white water" could be discharged on the water surface. By such a procedure the mill wastes would be carried downstream by the return surface current instead of upstream on the bottom to the water plant intake.

The mill's "white water" makes the entire Emory arm of the pool at times look like black ink instead of water; this color is very difficult to remove from the raw water.

The bacterial loads in the raw water at the water plant are naturally quite high, as the result of the sewage present in the water brought up to the filter plant by the density underflows. When the paper mill wastes are discharged on the surface, they are largely separated from the raw water, however, and the filter plant operator is able to obtain a chlorine residual and handle the bacterial loads much more satisfactorily.

As the fall of the year approaches, the water coming down the Emory River, as well as the shallow water in the upstream end of the Emory River embayment, is cooled. When the inflowing water from the Emory River becomes colder than that flowing down the Clinch River from Norris Reservoir, the inflow from the Emory headwaters then flows along the bottom of the reservoir past the Harriman water plant and proceeds downstream to the Clinch River. When this downstream flow occurs, an upstream current past the paper mill is produced on the water surface. This upstream current is caused by the cooling, sinking and flowing downstream along the bottom of water in the shallow upstream end of the Emory embayment. To replace this cooled water, an equal volume of water must flow upstream on the surface past the paper mill.

When this action occurs, it is necessary to change the discharge of mill waste from the surface outfall to the outfall at the river bottom, in order to prevent mill wastes from being carried up to the water plant on the surface. By this alternate use of low-level and high-level outfalls, the bulk of the paper mill wastes are always carried in a downstream direction. Due to the inevitable mixing action between the upper and lower moving layers of water, however, it is not possible to remove completely the paper mill waste from the water flowing upstream past the mill, either on the water surface or along the river bottom.

Control of Raw Water

Experience shows that, when the release of cold water from Norris Reservoir is as great as 6,000 cfs., the Emory River arm of Watts Bar Reservoir will become filled with Clinch River water throughout its entire length up to an elevation several feet above the permanent intake of the Harriman Water Plant. The maintenance, therefore, of a steady release of 6,000 cfs. from Norris during the summer months—an amount which approximately coincides with the normal operating schedule—supplies Harriman with Clinch River water relatively unpolluted by mill waste. During the years 1944 and 1945, a constant release of this magnitude was maintained from Norris Reservoir throughout the major portion of each summer. The Harriman filter plant operator has stated that the Norris releases materially reduced the color and taste of the raw water drawn into his plant.

Early in the summer of 1946, the management of the paper mill inaugu-

rated a system of treatment of its "white water." One result of this treatment was a materially improved raw water at the Harriman filter plant. In fact, the improvement was so marked during the summer of 1946 that it was no longer necessary to keep the surface water moving steadily downstream by maintaining a constant release from Norris Reservoir.

Occasionally, during the summer months, heavy rainfall on the Emory watershed produces a substantial inflow to the Emory arm of the pool. When this occurs, the relatively high velocity and volume of Emory River water forces the underlying Clinch River water downstream to some point below Harriman. As the Emory headwater flow recedes, however, the cool Clinch River water again moves upstream along the bottom and covers the permanent intake at the water plant.

When small increases in the flow of the Emory River occur in the summer months, the relatively warm inflow does not flush the Clinch River water from the pool cross-section at the water works intake but simply flows downstream on top of the cool Clinch River water, instead. Thus, a density current of a lighter liquid over a heavier one is produced. On such occasions, the Harriman filter plant operator usually takes advantage of the situation and uses a temporary surface intake in order to get the soft, relatively unpolluted Emory River water.

Under certain conditions of relative water temperatures, upstream density currents at mid-depth have occurred. Water in the downstream end of the Emory arm is then moving downstream on the surface, upstream at mid-depth, and downstream at the bottom. In other words, there are three separate and distinct levels of water movement in the cross-section. The mid-depth density currents rarely extend upstream as far as Harriman and so have little, if any, effect on the quality of the water at the location of the filter plant.

Before Watts Bar Reservoir was filled, no one realized that density currents would extend upstream into the Emory arm of the pool, a distance of thirteen miles. Numerous observations by the Hydraulic Data Division of the Tennessee Valley Authority, however, have shown such currents to exist every summer. Now that the physical forces that produce the density movements are recognized, they can be controlled to a certain extent, and the quality of the raw water available to the Harriman water plant can in some measure be improved.

New Filtration Plant at Midland, Pa.

By H. F. Holloway

A paper presented on Sept. 12, 1946, at the Western Pennsylvania Section Meeting, Pittsburgh, Pa., by H. F. Holloway, Midland Water Co., Midland, Pa.

THE Midland Water Co. supplies water to the Borough of Midland, Pa., a community of 8,000 persons located on the north bank of the Ohio River, approximately 37 miles downstream from Pittsburgh and 3 miles above the Ohio state line. The borough is highly industrialized, its principal plants being those of the Pittsburgh Div., Crucible Steel Co. of America, the Treadwell Construction Co., the Midland Barge Co. and the Mackintosh-Hemphill Co.

In March 1946 the water company placed in operation a new filtration plant to treat Ohio River water. The plant was designed by Morris Knowles, Inc., Engrs., and constructed by The Pitt Construction Co., both of Pittsburgh.

The new plant replaces an old fill-and-draw unit built in 1913 which consisted of three settling tanks, each with a capacity of 150,000 gal., and three filters with a total capacity of approximately 1.2 mgd. The old plant had been operated at capacity for the past several years, with no reserve for emergencies.

In 1940, the company considered constructing additional capacity at the old plant; later, modernizing the plant was discussed. Due to the high estimated costs, however, it was decided to construct a complete new plant near the center of the borough, rather than at the site of the old unit on the grounds of the Crucible Steel Co. works. The new plant was to have capacity to serve water to the borough, as well as the drinking water supply for the steel company. Final plans involved building a complete new filtration plant having a capacity of 2 mgd. and so designed that an additional 1-mgd. unit could be added at some time in the future, without major changes in the plant. The design also made provision for aerating or softening the raw water, as required.

Construction of the new plant was started in October 1944 and completed in March 1946, when it was placed in test operation. After about two weeks of intermittent runs, the new plant was placed in full operation and the old plant abandoned.

The new plant consists of two flash mechanical mixing chambers, a mechanical flocculating basin; a two-stage sedimentation basin, with the first stage or clarifier basin equipped with a sludge-removal mechanism; four gravity rapid sand filter units; a filtered water reservoir; pumping equipment; and other appurtenances.

Chemical Feeding and Mixing

Raw water is supplied to the plant through a 14-in. cast-iron main from

the existing auxiliary pump station on steel company property. The raw water is delivered to the two flash mixing chambers, each 6 ft. square with a water depth of 14 ft., a total capacity of 7,500 gal. and a detention period of approximately 5.5 minutes at a 2-mgd. rate. Each chamber is provided with motor-driven rapid mixing equipment, consisting of a centrifugal or propeller type impeller mounted on a vertical shaft. This provides turbulent mixing of the chemicals with the raw water. Overflow and drainage connections are provided, and the piping arrangement is such that the chambers may be operated in parallel or in series.

Chemicals for treating the water are added by four dry feed machines located in the chemical feed room. Two of these machines have a lime-feeding capacity of 400 lb. per hour, and the other two a capacity of 250 lb. per hour. The machines are adapted to feeding lime, alum, ferrous sulfate and soda ash. The chemicals can be applied: to the raw water in the flash mixing chambers; to the influent of the flocculation chamber; to the partially settled water between the mechanically cleaned clarifier basin and the sedimentation basin; approximately three-fifths of the way through the sedimentation basin; and to the filtered water in the clear well. There is an additional dry feed machine for feeding activated carbon to the partially settled water between the clarifier basin and the sedimentation basin, at a point three-fifths of the way through the sedimentation basin, or at the inlet pipe to the filters. The feed can also be split to the several points of application, if desired.

The flocculation basin consists of a concrete chamber with an effective length of 31 ft., a width of 20 ft. and a water depth of 14 ft., which gives it a capacity of 64,800 gal., or a detention period of 46.6 minutes at the 2-mgd. rate of flow. This basin is equipped with slow mixing paddles, operating parallel to the flow of water. The outside diameter of the paddle assembly is approximately 12 ft., and the tips of the paddles move at a maximum speed of 1.5 fpm. Wooden baffles are placed between each of the three paddles to prevent short circuiting.

The flocculated water enters the first stage of the sedimentation or clarifier basin. This basin is approximately 60 ft. long and 20 ft. wide, with a water depth of 14 ft. The capacity is 126,000 gal., providing a detention period of about 90 minutes at a 2-mgd. rate. The basin is equipped with dragchain type sludge-collecting equipment, the sludge being removed to a hopper or collecting compartment at the inlet end of the basin. The accumulated sludge is discharged into the storm sewer.

The effluent from the clarifier basin discharges into the sedimentation basin by means of a pipe and thence through a perforated baffle wall to secure good distribution. The sedimentation basin has an effective length of 90 ft. and a width of 25 ft., with a water depth of 14 ft., giving it a capacity of approximately 240,000 gal. This is equivalent to a settling period of 2 hours and 53 minutes at the design rate of flow. Perforated baffles are at each end of the basin, extending from the top to the bottom of the basin, and have 2-in. diameter holes, spaced approximately 15 in., center to center. The basin is equipped with two plug valves for draining.

Filters and Controls

The settled water discharges to four 500,000-gpd. gravity rapid sand filters.

each having a sand area of 196 sq.ft. The filters, two of which are located on each side of a pipe gallery, are equipped with modified "Wheeler" bottoms of pre-cast concrete units, securely anchored by steel rods to the reinforced concrete filter floor. There are 16 in. of graded gravel and 27 in. of sand, and surface wash equipment is installed in each filter. The filter walls are lined with white glazed tile, which permits easy cleaning and adds to their attractiveness.

The operating tables are constructed as an integral part of the parapet wall in front of the filters. The controls for the hydraulically operated filter valves, the surface wash water valves, and the gages indicating the loss of head and rate of flow are located on the tables.

In addition to a Venturi type controller for each filter, a master controller has been provided to operate with the individual controllers of each filter when the clear well is filled.

Wash water for the filters is furnished by two 2,700-gpm. motor-driven centrifugal pumps, designed to operate against a head of 50 ft. The wash water line is provided with a pressure relief valve in order to control the water pressure and to insure against pressures in excess of that desired for washing. The wash water is taken directly from the clear well.

The clear well or filtered water basin has a capacity of approximately 176,000 gal. and is located beneath the filters, pipe gallery and pump room. This basin has capacity to store filtered water for a period of 2 hours and 8 minutes at the designed rate of 2 mgd. The clear well is designed and baffled to permit the filtered water to be chlorinated at the inlet of the baffled section, thus insuring a maximum of chlorine contact time.

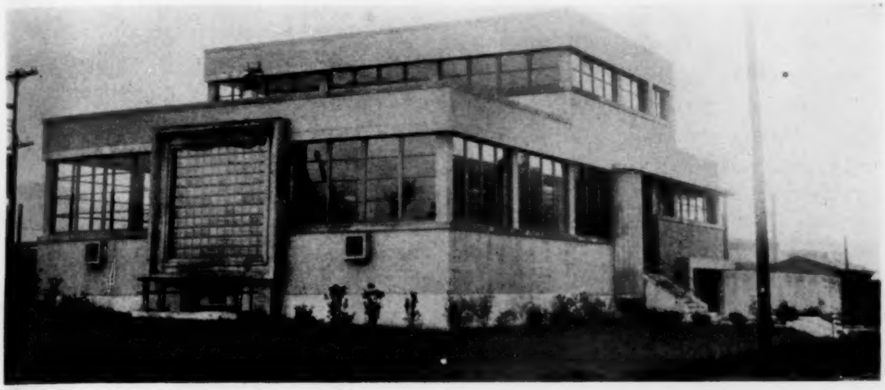


FIG. 1

Duplicate chlorinators are provided and either or both may be used to apply chlorine to the incoming raw water, the influent to the secondary settling basin or the clear well. Ammonia can also be added in the clear well, approximately 6 ft. from the point of chlorine application.

Pumps and Structures

The pump room is located adjacent to the filters and pipe gallery. The wash water pumps are in the southern part of the room and the high-lift

pumps in the northern part. The high-lift pumps are motor-driven centrifugals, each having a capacity of 1,400 gpm. Automatic priming equipment maintains the high-lift and wash water pumps in operating condition, making it impossible to start a pump unless it is properly primed. A unit will also be shut down if it should lose prime. Vacuum is maintained through a tank by means of an electric-driven pump. If the pump should fail, a water ejector cuts in at slightly less vacuum and maintains it until repairs are made to the pump.

The plant superstructure has been designed to present an attractive appearance (Fig. 1). It houses the offices, filter operating room, electric control room, chemical feed room, chlorination room, chemical storage, laboratory, shops and garage. The filter operating corridor and the main entrance corridor join at the sight well to the clear well. The sight well extends about 3 ft. above the floor at the intersection of the corridors and a submerged light illuminates the filtered water. Adjacent and with entrance to the operating floor is the superintendent's office. A separate business office is connected with the front corridor. The superintendent's office houses the remote control water-level recorder, the recorder for the booster station supplying water to Midland Heights, and the Venturi recorder that measures and records the plant output. Opposite the superintendent's office is the electric control room, which houses the control panels, the clear well gage, pressure gage and remote water-level indicator. At the rear of the electric control room is the chlorination room, which has ready access to the loading platform. The chemical feed room houses the dry feed machines and is separated from the rest of the operating plant and is adjacent to the laboratory and workshop.

The laboratory is designed for routine chemical analysis and is equipped with a laboratory table, sink and sampling table that permits securing water samples from various parts of the plant.

The chemical storage room is above the chemical room. The chemicals are added to the dry feed machines by means of hoppers which extend through the storage room floor. The chemicals are handled by an electric hoist and monorail equipment.

A meter testing room and workshop have been provided, as well as a two-car garage. This arrangement is very advantageous, as all departments are under one roof. Shower and locker rooms have also been provided adjacent to the workshop.

Experience With Plant

The plant has been operating now for a number of months and there have been few difficulties experienced, beyond the usual troubles of initial operation.

As the source of raw water is the Ohio River, the plant has been designed for a water that has a varying chemical character, high bacterial loading, at times, and tastes and odors that must be removed.

Alum is applied to the two flash mixing chambers; the amounts used having varied from as much as 25 ppm. to as low as 3 ppm., depending on the pH and turbidity of the raw water. The average dose is 9 ppm. Low turbidities of 3 to 8 ppm. are usually accompanied by low pH values of from 5.0 to 6.2, and for one period in June only 3 ppm. of alum was used

Turbidities averaging 25–35 ppm. usually are handled with about 8 ppm. of alum. Credit for this is due mainly to flash mixing and the type of flocculation basin. The turbidity of the raw water averages 20 ppm.; of the primary settling basin, 10 ppm.; and of the water applied to the filters, 4 ppm. Roughly a third of the lime is added between the flash mixing chambers and the flocculating basin, the remainder being added between the clarifier basin and the sedimentation basin, producing an average pH of 6.9 at this point. The initial pH of the raw water is 6.3. The average lime dose is 13 ppm., and the finished water has a pH value of 8.6. Activated carbon has been used successfully in removing the slight objectionable taste which has developed at times. An average dose of 5 ppm. of carbon is used.

Although the cost of this plant came to 50 per cent more than the 1944 estimate, the quality of water obtained has been good and has been worth the additional investment. The physical appearance of the plant makes it a welcome addition to the borough. The grounds have been landscaped, making the surroundings very attractive. Customers paying their bills are invited to inspect the plant and, if time permits, are escorted through the plant while its operation is explained to them. The comments are usually very favorable. As the majority of visitors have no conception of the source or kind of treatment given their water, they leave understanding that a water supply requires more than just heavy rain.

Acknowledgment

It would be an injustice not to extend due credit for the construction of the plant, details of design and the satisfactory operating results thus far obtained to C. H. Young, Dist. Engr. of the Dept. of Health, in whose district the plant is operated.

Discussion

M. A. Slone

Div. Engr., Morris Knowles, Inc., Cons. Engrs., Pittsburgh, Pa.

As the Midland plant has been in actual operation for such a short time, it is of course to be expected that more troublesome raw water conditions will prevail than have yet been experienced and that the author's most severe headaches are yet to come. The increasing familiarity of the plant personnel with operating details, however, and the knowledge being gained of how best to cope with extreme bad water conditions in the river, make it appear that a good quality of water will be produced continuously, regardless of the raw water quality.

The successful operation of any plant, and the results produced thereby, depend upon the quality of the supervision and the caliber of the operating force. Although the author himself has had years of experience in operation of water utilities and water treatment plants, the operators employed at the plant had had no experience in the field. One came up from the ranks, and all have shown exceptional interest and ability to learn the fundamentals. The laboratory tests are carried out by the operators, so that they not only make the tests for control of operation, but actually put these controls into practice.

As Midland is located approximately 37 miles down the Ohio River from

Pittsburgh, it is not difficult for any water works man, although unfamiliar with the actual raw water conditions, to imagine the quality of the water at the intake to the plant. Sewage and industrial wastes from many municipalities and industries on the Monongahela and Allegheny watersheds, as well as the Ohio watersheds below Pittsburgh, are discharged into the streams. As a result, the water not only is highly contaminated but has extreme variation in chemical quality. This variation in quality is reflected by the range experienced in pH, alkalinity, hardness, turbidity, and tastes and odors.

During the war, a large synthetic rubber plant was constructed by the government a few miles above Midland, and at the peak of operation about 120 mgd. of industrial water was used, most of which was returned to the river. This added materially to the problem of operation of the plant at Midland, particularly in the removal of tastes and odors. Before the new plant was placed in operation, however, the butadiene part of the rubber plant at Kobuta was practically shut down, leaving only the styrene part in operation. This curtailed the use of water and its subsequent discharge back into the stream to about 60 mgd. Although these wastes are discharged on the opposite side of the river from the intake at Midland, their effects are felt, and careful controls—for tastes and odors in particular—are necessary at all times to produce a water which will be acceptable to consumers.

Values of pH as low as 5 have been experienced, and the hardness varies from 60 to 250 ppm. Alkalinity has varied from 1 to 33 ppm. since the plant was placed in operation, and turbidity from 2 to 65 ppm. Much higher turbidities of course are to be expected during times of high water. The manganese content of the water has varied from 0.5 to 1.3 ppm.

Because of this extreme variation in the quality of the raw water, it was necessary in the design and construction of the plant to provide greater flexibility of treatment than is normally required. Also, because of the wide variation in hardness, provisions have been made for softening, when and if required. Up to the present time, however, softening has not been inaugurated. In the design of the plant, provisions were made for economical addition of aerators, should this treatment of the raw water prove necessary to secure satisfactory results.

In general, the plant is conventional, but it provides for greater flexibility in the treatment operation than is normal in a small plant. The different stages of treatment and the mechanical equipment provided are normally not to be found in plants of this size, but are common to large plants. The chief endeavor of the water company and its parent company, the Pittsburgh Div. of the Crucible Steel Co. of America, following the decision to build a new plant, was to provide one that would be a credit to the community, one that would produce a finished water of high quality at all times, regardless of the quality of the raw water, and one that would operate economically, so as to keep water costs to a minimum. As a large percentage of the consumers are employees of the steel company, the facilities required to produce a water of high quality at a minimum cost have been provided in the plant without consideration of the initial cost of construction.

As Midland is an industrial community, it was also deemed necessary by the company to make provision for a comparatively large increase in the use of water. Consequently, provisions have been made for increasing the capacity of the plant to 3 mgd. merely by the addition of two filter units and a third sedimentation basin. This can be done economically and without much delay when consumption requires additional capacity.

Two flash mixing chambers, having a total detention period of 5.5 minutes at a 2-mgd. rate, have been provided. With the two chambers and the piping arrangement, softening can readily be carried out not only for a plant of 2 mgd. capacity, but also for the additional 1 mgd.

The detention period in the flocculating basin is 46.6 minutes at the 2-mgd. rate, or more than 30 minutes at a 3-mgd. rate. As the author has stated, the paddles operate parallel to the flow, making it possible to place the motors driving the paddles in the pipe gallery adjacent to the flocculating and sedimentation basins.

The present clarifier and sedimentation basins provide for a total detention period of 4 hours and 23 minutes at a 2-mgd. rate. An additional basin will be required when the capacity of the plant is increased to a total of 3-mgd.

A clear water well with a storage capacity of 2 hours and 8 minutes at a 2-mgd. rate has been provided underneath the filters, pipe gallery and pump room. The water from the clear well is pumped directly into the system. The clear well has been baffled so that chlorine and ammonia can be applied at points providing approximately two hours' chlorine contact time before the treated water is pumped into the system for distribution.

Provision has been made for pre- as well as post-chlorination—both of which are now used—and for free residual chlorination if later required for taste and odor treatment. Residuals of 0.2 to 0.25 ppm. are maintained in the water going to the filters and residuals of 0.1 to 0.15 ppm. in the water discharged to the distribution system.

Although provisions have been made for the use of various coagulation chemicals, only alum and lime have been applied up to the present time. The use of ferrous sulfate and soda ash may prove desirable later, as the plant experiences a wider range of quality of raw water, but so far excellent results have been obtained with the use of only alum and lime.

During the comparatively short time that the plant has been in operation, best results have been obtained by applying approximately one-third of the dosage of lime between the flash mixing chambers and the flocculator basins, and two-thirds between the clarifier basin and the settling basin.

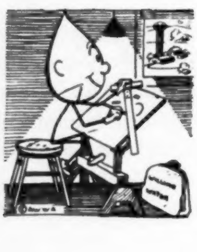
Activated carbon has been found necessary and is used to eliminate tastes and odors. The dose has averaged 5 ppm. and is now being added at the influent chamber of the sedimentation basin.

Excellent results have been obtained so far in the bacterial and chemical quality of the filtered water. This accomplishment has been made despite the facts that the plant is little beyond its trial period, that the operators had to be trained for their duties and that little was known of the wide variation in the daily quality of raw water until the new laboratory was placed in operation and tests could be made. The

finished water has been free of *Esch. coli,* and the bacterial counts have been very low.

The absence of turbidity and color, the satisfactory reduction of iron and manganese, elimination of tastes and odors and satisfactory pH control of the water have all been made possible by the flexibility in the design of the plant, the mechanical facilities that are provided, and the designed periods of treatment allowed. Regardless of the quality of supervision and operation of a plant, satisfactory results cannot be accomplished without the necessary facilities.

Many difficulties were experienced during the construction of the plant due to shortages in materials, delays and strikes. Nevertheless, a much needed plant has been provided and is now in operation—to the credit of the company and to the benefit of the community. Actual costs of construction were necessarily high, but this did not deter the company from providing necessary and adequate facilities to give the citizens of Midland water of high quality. Much credit is due the company for inaugurating and carrying out the project during a period of high prices and difficulties in construction.



Ground Water Studies in Wisconsin

By F. C. Foley, W. J. Drescher and G. E. Hendrickson

A paper presented on Nov. 16, 1946, at the Wisconsin Section Meeting, Green Bay, Wis., by F. C. Foley, Geologist, W. J. Drescher, Engr., and G. E. Hendrickson, Geologist, all of the Div. of Ground Water, Water Resources Branch, U.S. Geological Survey, Madison, Wis.

GROUND water investigations have been carried on in Wisconsin for many years by several state agencies. The Wisconsin Geological Survey (WGS), under the direction of E. F. Bean, State Geologist, and under the immediate supervision of F. T. Thwaites, has collected well samples and prepared well logs since 1912. During the past two years more than 7,500 well samples from 109 wells have been examined. Well drillers throughout the state have furnished approximately 61,000 individual samples from many hundreds of wells for the files of the WGS. The geologic information and water level and pumpage data on file have been and will be of very great value to the current detailed ground water studies (1).

Previous Studies

Among existing sources of information are the regional studies of water resources made by the WGS and the U.S. Geological Survey (USGS) during the early years of the century (2).

The Bureau of Sanitary Engineering of the State Board of Health has been interested in ground water supplies, not only from the viewpoint of sanitation and chemical problems, but also because of the problems of supply and conservation of ground water. These records are excellent and will be very valuable in any investigations.

An investigation of the ground water resources of Milwaukee County has been made by the Milwaukee County Regional Planning Commission. A report on that investigation is now being prepared.

The Soil Conservation Service of the U.S. Dept. of Agriculture has done some investigation of ground water in its soil erosion studies. A number of observation wells were established in 1934, in 8 of which water-level measurements are still being made by the USGS.

The Wisconsin State Conservation Commission has done considerable exploration of shallow ground water supplies as sources of water for fire fighting and in connection with drainage areas. It still measures water levels in 5 wells.

In 1944, 10 observation wells were established by the USGS in the northern Wisconsin River Valley. Measurements of water levels in that area are being tabulated by the Wisconsin Valley Improvement Co., as well as by the USGS.

Initiation of Current Studies

For many years artesian wells have supplied water for municipalities, industries and domestic users in Wisconsin. A few of these wells were drilled prior to 1875. By 1900 most of the larger industries of the eastern

part of the state were using artesian well water, and municipal supplies in the east and south of the state were obtained principally from artesian wells. The greatest concentration of artesian wells and the heaviest pumpage are in the eastern part of the state, particularly in the Milwaukee-Waukesha area. There are other local areas of heavy pumpage in the larger municipalities. Water levels had been declining for many years in the heavily pumped artesian areas prior to 1942. Increased use of water during the war years accelerated the rate of decline, so that it became necessary to lower pump settings in many wells. The increasing demand and accelerated decline caused concern among ground water users in the state, and it became apparent that more should be known about Wisconsin's ground waters.

As a result of this situation, a bill was introduced in the 1945 legislature by the Joint Committee on Finance, making an appropriation to the Board of Regents of the Univ. of Wisconsin, "for the purpose of investigating the underground water resources of the state, determining the present use and depletion thereof and recommending to the legislature such action as may be deemed necessary to conserve these underground water supplies as a public resource." The bill authorized the university "to co-operate with the appropriate agencies of the federal government in conducting such study." A Co-operative Agreement was signed on Jan. 15, 1946, with the USGS, to conduct ground water studies in Wisconsin. The senior author was placed in charge of the federal part of the investigation. The university is represented by a committee consisting of the State Geologist, as Chairman, and two members of the University faculty.

The geologist in charge arrived in Madison on February 16, 1946, to start the investigations. The delay in starting was due in a large part to a shortage of qualified personnel. Many technical men of the USGS were still on active duty with the armed forces. At the time, however, an office was set up, adjacent to the office of the State Geologist, in the university.

The permanent staff now includes the authors and a clerk-stenographer. During the summer months, three Univ. of Wisconsin civil engineering students worked full time on the project and one is continuing on a part-time basis during the school year.

Procurement of necessary equipment has been difficult. Delivery has been very slow on almost everything that has been ordered, but most basic equipment is now on hand and in use. Transportation was a bottleneck for a time, but one Ford panel truck has been loaned by the Topographic Branch of the USGS and two half-ton pickup trucks have been procured from Army surplus. Sufficient steel tapes for basic needs have been procured and more are on order. Water-stage recorders ordered last spring have recently been received, and ten have been installed. A great deal of difficulty has been encountered in obtaining the small amount of materials necessary for construction of shelters for the recorders where installations must be in the open.

Geology of Wisconsin

For an understanding of the problems involved, it is necessary to review briefly the geology of Wisconsin (3), for the occurrence of ground water and studies relating to it involve basically geologic concepts.

The rock formations of Wisconsin include pre-Cambrian igneous and

metamorphic rocks, Paleozoic sandstones, limestones and shales, and Pleistocene deposits of sand, gravel, and boulder clay of glacial origin.

The pre-Cambrian rocks are at the surface or are covered only by glacial drift in a large area in the north and north-central parts of the state. To the east, south and west of this area, they are covered by Paleozoic strata which dip generally away from the area. The pre-Cambrian formations are of low permeability and do not yield large quantities of water, but in some places furnish enough for domestic use.

Cambrian sandstones crop out to the east, south and west of the exposed pre-Cambrian. The outcrop area is greatest in the south and west sections of the state. The Cambrian sandstones are the most important artesian aquifers in the state, and in the outcrop area they are also an important source of ground water under water-table conditions.

The Lower Magnesian limestone overlies the Cambrian sandstones and occurs at the surface or directly under the glacial drift in a nearly continuous, concentric band around the east, south and west of the exposed Cambrian sandstones. Over a large area in the western part of the state, the Lower Magnesian limestone caps the hills with the underlying Cambrian sandstones exposed in the stream valleys. The Lower Magnesian furnishes water to shallow wells over part of its outcrop area. Where it occurs as a thin capping on high hills, it generally lies above the water table.

Between the Lower Magnesian limestone and the overlying St. Peter sandstone is an erosional unconformity. Accordingly, both the Lower Magnesian and the St. Peter range considerably in thickness, and in some places the Lower Magnesian is entirely absent.

The area in which the St. Peter sandstone lies at the surface, or directly under the drift, is relatively small. It occurs as small patches overlying the Lower Magnesian uplands in the west and crops out in valleys in the south and southwest. It also forms a very narrow band from a point west of Marinette southward to the state line just east of Beloit. It is an important water table aquifer where it crops out in the valleys, and in the eastern part of the state it is a source of artesian water.

The Galena-Platteville limestones overlie the St. Peter sandstone and are exposed at the surface, or under the drift, as a continuous broad belt from Marinette County southward to the state line, and form the upland surface in the south and southwest parts of the state. Relatively small patches occur, capping the St. Peter sandstone in the northwest and west. The Galena-Platteville limestones form an important source of ground water in the outcrop area in the southwest part of the state. In some areas, the water table in the Galena-Platteville is perched.

The Richmond shale crops out as a narrow continuous band southwestward from Green Bay, along the east shore of Lake Winnebago, and southward to the Illinois state line near Walworth. It also occurs as minor patches capping the Galena-Platteville uplands in the south and southwest. The shale is important as a confining formation between the overlying Niagaran limestone and all aquifers below it, but in itself it is of no importance as an aquifer.

The Niagara limestone overlies the Richmond shale in the area adjacent

FIG. 1. Observation Wells in Wisconsin

to Lake Michigan. It extends from Door County southward to the state line in Kenosha County. It is important as a source of both water table and artesian ground water. Most of the few present-day flowing wells in eastern Wisconsin derive their water from the Niagara. In the heavily pumped sections of the Milwaukee area, where deep wells are open in both the Niagara and in the underlying sandstones, the Niagara is losing water to the sandstone formations.

The Milwaukee formation occurs as a narrow band along the Lake Michigan shore from Sheboygan to Milwaukee. It is composed of limestone and interbedded shale, and is of minor importance as a source of water.

The sand and gravel of the glacial drift is an important source of ground water over most of the glaciated area.

With the recent decline in water level in deep wells in certain areas, the possibility of developing supplies from the glacial drift is receiving increasing attention. Because of the considerable variation in character of the glacial drift, even in small areas, it is generally necessary to drill test wells prior to the drilling of a producing well. In the area of pre-Cambrian outcrop the mantle of glacial drift is, with few exceptions, the only possible source of ground water.

Observation Well Program

A series of observation wells, which will eventually cover the entire state, is being established. Measurements of water levels are made in the observation wells at intervals, usually a week or a month. Wells are selected so that, as far as possible, each type of ground water occurrence is represented. Water-level data obtained in order to show fluctuations as a result of climatic conditions, water use and drainage are valuable in obtaining an over-all picture of ground water resources. In detailed studies, water-level data are essential in determining the effects of pumping and seasonal trends, and for determining drawdown and recovery during pumping tests.

Wherever possible, unused wells are set up as observation wells, but some of the shallow wells have been driven by hand, using well drive points, for specific use as observation wells.

As of Oct. 15, 1946, 108 observation wells had been established in Wisconsin. Location of the observation wells is shown in Fig. 1. Not all individual wells in the Milwaukee area are shown, as they are too closely spaced to appear satisfactorily. The group of 11 wells in the Coon Creek area in Vernon and Monroe counties was established in 1934 by the Soil Conservation Service of the U.S. Dept. of Agriculture and was later taken over by the USGS. Records of water levels in the wells have been published in USGS Water Supply Papers **777, 817,** 840, 845, 886, 908, 938, 946 and 988. Water levels in 1943 are the latest to be published, but those for later years will appear in due course, both for the Coon Creek area wells and for more recently established wells.

The wells in the northern Wisconsin River Valley were established in 1944. All others have been established since the current investigations were started. Five wells being measured by the Wisconsin State Conservation Commission are not shown on the map. They are all in northern Wisconsin.

Measurements of water levels are made by members of the USGS staff, state employees, local observers—usually a person on whose land the well is located, or someone living nearby—and by municipal water departments. Measurements are made with a steel tape from a fixed measuring point and are taken to 0.01 ft.

Water level measurements in one key well, on which data are available since 1934, refute the belief held by some people that water levels generally have declined drastically in Wisconsin during the past several years. The well is a water table well, 44 ft. deep, located 2½ miles southwest of Cashton in Monroe County. Measurements were started on June 29, 1934, at which time the water level was 17.76 ft. below land surface datum. The water level was at its lowest point during the more than 12 years of record on Feb. 28, 1935, at which time the water level was 18.71 ft. below land surface datum. The well started to recover in 1935 and reached the

highest level during the period of record on Mar. 29, 1946, when the water level was only 6.10 ft. below land surface datum.

Water Levels in Municipalities

The Bureau of Sanitary Engineering of the State Board of Health has started a valuable project to obtain records of quantity of water pumped and static and pumping water levels in

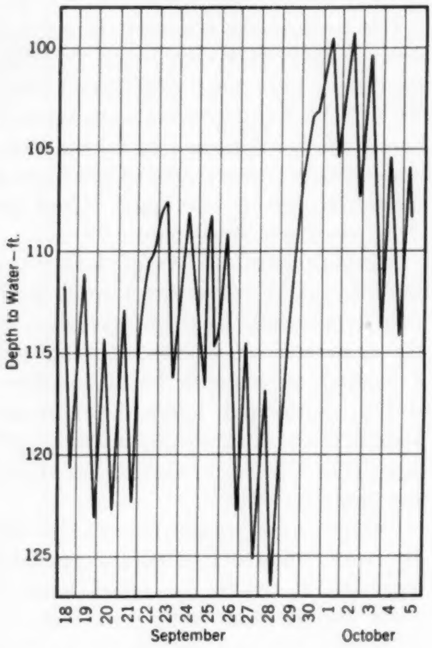


FIG. 2. Automatic Water-Stage Record

municipal wells in Wisconsin. Basic data and monthly report forms have been printed by the bureau.

The basic data sheets are for records of basic information on each well in a municipality, used or unused. The report forms, submitted monthly, are for a daily report of quantity of water pumped in each well and for a report of static and pumping water levels at intervals, preferably at least once a week. The forms are submitted in duplicate by municipal water superintendents, with one copy retained by the bureau and one copy going to the USGS.

Report forms have been sent to 141 municipalities in Wisconsin, and it is planned to complete the coverage of the state. Data compiled from the reports will furnish a very valuable record of water used in Wisconsin and a continuing check on trends of water levels. Many municipalities have kept records of pumpage and water levels for years, but many more have started keeping records as a result of the project. The records are of value to the municipalities as well as to the understanding and solution of the state's ground water problems. The information will be of increasing value as the period of record becomes longer. Reports are being returned, though not all the municipalities have reported as yet.

Eastern Wisconsin Area

Work is being concentrated at present in eastern Wisconsin, where the decline in water levels in deep artesian wells has caused greatest concern. The investigation is a detailed study of the water resources of the area, not only of the deep artesian aquifers but also of shallow sources. Measurements of water levels, both static and pumping, are being made to determine trends and to map the piezometric surfaces. Pumping tests have been made and others will be made to determine the hydraulic characteristics of the various aquifers. Studies of past and present quantities of water pumped and the uses to which the water is put are in progress but are not yet far enough advanced to give total quantities of ground water being withdrawn. Recharge areas and rates of recharge will be investigated. It is planned that the

completed study will show present and future effects of withdrawal of ground water on water levels, the interference that can be expected between wells, and the quantities of water that can be withdrawn to develop the ground water resources to their fullest extent for perennial use.

Twenty wells have been established as observation points in Milwaukee County, where decline in water levels is most widespread. Most of these wells are unused, but a few that are being pumped intermittently are being measured. Observation wells have been established also at many points outside Milwaukee County, as shown in Fig. 1.

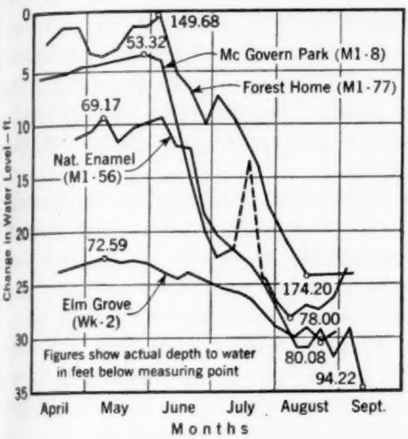


Fig. 3. Hydrographs of Decline in Water Level of Four Wells

Automatic water-stage recorders have been placed on eight wells in eastern Wisconsin. More will be set up as soon as recorder shelters have been constructed. All those now in operation are in pumphouses or buildings where individual shelters are not necessary. As an illustration of the record provided by an automatic recorder, that of the Milwaukee *Journal* well, located at 333 West State St., Milwaukee, for the period Sept. 18 to Oct. 5, is shown in Fig. 2. Daily fluctuations ranging from about 7 to 12 ft. are shown. The well is in an area of heavy pumpage where nearby wells are used largely for air-conditioning. It is interesting to note the recovery in static water level on Sunday, Sept. 29, when pumpage was reduced. It is apparent that spot measurements of water levels weekly or even daily do not give a real picture of conditions in a well where fluctuations are large and rapid.

Observation wells were first set up by the USGS in eastern Wisconsin in April 1946, and additional wells have been established during succeeding months. The period of record is very short, and little can be determined yet regarding the present trend in water levels. Static levels in four of the Milwaukee area wells for which the period of record by the USGS is longest are shown in Fig. 3. Water levels in three of the wells reached highest points in May, whereas the Forest Home Cemetery well reached its peak in early June. Lowest points were reached in August and September.

The greatest range in water levels measured in 1946 was in the Milwaukee *Journal* well, for which only an 18-day record is shown in Fig. 2. Weekly measurements with a tape were started on April 23, 1946, before

TABLE 1

Greendale Pumping Recovery Tests

Well	Duration	Coefficient of Transmissibility, T
	hours	*gpd./ft.*
Ml-91	9*	21,600
	14½	14,100
Ml-92	17	16,400
	17	16,100

* Drawdown.

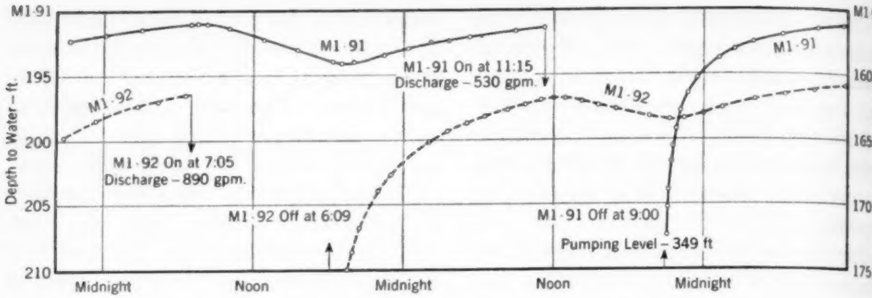


FIG. 4. Recovery After Pumping and Interaction of Wells

a water-stage recorder was available. The highest water level measured was on May 9, when it was 74.02 ft. below the top of the casing, and the lowest was on Aug. 22, when it was 121.29 ft.—a range of 47.27 ft. On Oct. 2, the high point shown in Fig. 2 was 99.23 ft. below the top of the casing.

It is expected that water levels will rise during the winter months, due to normal decrease in quantities of water pumped. Probably the highest levels in 1946 were reached in May or June, shortly after measurements were started. It is evident that measurements must be continued indefinitely to show long-term trends.

Pumping Test Coefficients

The amount of water that can be withdrawn from an artesian aquifer depends upon the amount of water which is absorbed by that aquifer in the area of outcrop, upon the ability of the aquifer to transmit water to the wells and upon the amount of water that is released from storage in the aquifer with a reduction in artesian head. The rate and amount of decline of water levels, due to pumping from an aquifer, depend upon the transmissibility and storage capacity of the aquifer.

The ability of an aquifer to transmit water is expressed as a coefficient of transmissibility, which is defined (4) as the number of gallons of water that will move in one day through a vertical section of the aquifer one foot wide and of a height equal to the full thickness of the aquifer, with a hydraulic gradient of 100 per cent.

The storage capacity of an artesian aquifer is expressed by a coefficient of storage, which is defined (4) as the volume of water, measured in cubic feet, released from storage in each column of the aquifer having a base of

TABLE 2

Greendale Pumping Interference Tests

Obs. Well	M1-91		M1-92	
Pumped Well	M1-92		M1-91	
	ON	OFF	ON	OFF
Duration, *hours*	11	17	10	14½
Coefficient of transmissibility *T*, *gpd./ft.*	35,100	32,300	21,800	20,700
Coefficient of storage, *S*	0.00047	0.00042	0.00031	0.00027

one square foot and a height equal to the thickness of aquifer, when the artesian head is lowered one foot.

In order to determine the coefficients of transmissibility and storage of the artesian aquifers that underlie most of eastern Wisconsin, pumping tests have been made at Greendale, Town of Lake, McGovern Park in Milwaukee County, Badger Ordnance Works, and at Waukesha. The Waukesha test was the most extensive but was made too recently for results to be included in this report. Other tests will be made throughout the state.

TABLE 3

Average Greendale Pumping Coefficients

No. of Wells	No. of Tests	Avg. Duration *hours*	Coeff. of Transmissibility T in *gpd./ft.*			Coeff. of Storage S		
			Max.	Min.	Avg.	Max.	Min.	Avg.
Recovery and Drawdown								
2	4	14½	21,600	14,100	17,000			
Interference								
2	4	13	35,100	20,700	28,000	0.00047	0.00027	0.00037
All Tests								
2	8	13¾	35,100	14,100	22,300			
2	4	13				0.00047	0.00027	0.00037

Greendale Tests

A typical series of pumping tests was made of the St. Peter and Cambrian sandstones in June 1946 at Greendale in Milwaukee County. Both of the municipal wells used in the test are cased through the Richmond shale, thus shutting off all water except that which comes from the sandstones. Each well in turn was pumped at a constant rate and the fluctuation of the water level in the idle well was measured. The pump was then turned off, and the amount and rate of recovery of the water level in each well were measured to the nearest 0.01 ft. by means of a steel tape. Airline measurements of pumping levels were made in the north well but could not be made in the south well, due to mechanical difficulties. Figure 4 shows hydrographs of the recovery of each well after pumping and the effect of each well on the other. The distance between the two wells is 2,110 ft.

Certain features of this test that differ from the ideal test should be pointed out. Wherever possible, water levels should be measured in at least two observation wells other than the pumped well; at Greendale only one observation well was available. Before a well is turned on or off during a test it is desirable that the water levels be at equilibrium; that is, that the water level in all wells should be rising or falling at the same rate. It

was impossible, at Greendale, except on the last day of the test, to allow the water levels to reach equilibrium because of the necessity of meeting the municipal demand for water.

The non-equilibrium formula (5, 6) which was developed under the direction of Charles V. Theis of the USGS was used to determine the coefficients of transmissibility and storage. The formula is:

$$s = \frac{114.6Q}{T} \int_{\frac{1.87 r^2 S}{Tt}}^{\infty} \frac{e^{-u} du}{u}$$

in which s is the drawdown, in feet, at any point in the vicinity of a well pumped at a uniform rate; Q is the discharge of the well in gallons per minute; T is the coefficient of transmissibility, in gallons per day per foot; r is the distance, in feet, from the pumped well to the point of observation; S is the coefficient of storage of the aquifer; and t is the time, in days, that the well has been pumped.

In making use of the formula, it is assumed that the aquifer is infinite in extent, that its transmissibility is the same at all places, that it is confined between impermeable beds above and below, that the coefficient of storage is constant and that water is released from storage instantaneously with a decrease in artesian head. In determining coefficients the non-equilibrium formula may be applied in three ways: (1) to the drawdown or recovery of at least two observation wells at any time, (2) to the amount and rate of drawdown or recovery of a single observation well or (3) to the rate of recovery of a pumped well after pumping ceases.

Methods (2) and (3) were used in applying the formula to the Greendale tests. Tables 1–3 give the coefficients of transmissibility and storage obtained from the Greendale tests.

Application of Coefficients

The curves in Fig. 5 were computed by means of the non-equilibrium formula, using the average coefficients obtained from the Greendale tests. The curves represent the theoretical drawdowns in water levels at various distances from a pumped well at the end of 6 months, 1, 3, 5 and 10 years, produced by continuous pumping from the well at the rate of 500 gpm.

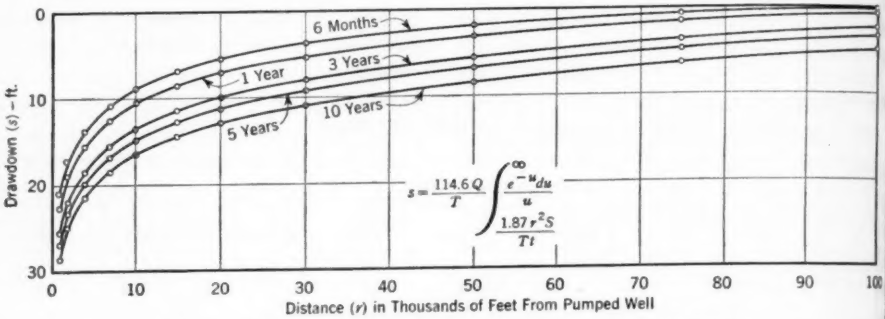


Fig. 5. Theoretical Drawdown Curves for Greendale Wells

As the non-equilibrium formula assumes an aquifer of infinite areal extent, the drawdown figures obtained from these curves must be corrected for the effects of lateral boundaries and changes in thickness and character of the aquifer before they are applicable to the sandstones at Greendale.

The lateral boundaries of the sandstones in the Milwaukee-Waukesha area are the outcrops of the sands and possibly pre-Cambrian highs to the northwest on a line between Waterloo and Fond du Lac. The effect of the outcrops of the aquifers after a long period of continuous pumping is to establish a new distribution of artesian head such that water withdrawn from the formations is replaced by recharge or by water drawn from storage in and near the outcrop area. Boundaries such as the pre-Cambrian highs within the formation, which restrict the capacity of the formation as an aquifer, have the same effect as reduced permeability and cause an increased rate of decline of water levels. The magnitude of the effect of the pre-Cambrian highs is as yet unknown.

Conclusion

A good start has been made on the study of the ground-water resources of Wisconsin, but very much remains to be done. The detailed investigation of the eastern artesian area alone, on which work is now being concentrated, will require at least two years. After a report has been made, measurements and records should be maintained indefinitely. Water supply in the remainder of the state presents a series of interesting and important problems, the solution of which will require a great deal of work.

Acknowledgment

The current ground water investigations are based on co-operation between federal, state, local and private personnel, ranging from the formal co-operative agreement between the university and the USGS to the collection of data from owners of private wells. The co-operation in Wisconsin has been splendid and complete. The State Board of Health is making its extensive records available and is actively co-operating in collecting basic data. The Milwaukee County Regional Planning Commission has made available data collected during its study of ground water conditions in Milwaukee County—data very valuable in connection with the current investigations. All municipal officials as well as private corporations and individuals have readily assisted in the collection of data and have allowed access to wells and well records. The co-operation is particularly appreciated in connection with pumping tests where disruption of normal operating schedules has been necessary.

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Well Cleaning and Rehabilitation

By H. P. Wenrick

A paper presented on Oct. 24, 1946, at the California Section Meeting, San Francisco, Calif., by H. P. Wenrick, Engr., Associated Engineers, Palo Alto, Calif.

THE necessity for cleaning or rehabilitating a well presumes some failure of the well, resulting either in production of unsatisfactory water or in decreased yield. Water may be unsatisfactory due to: (1) its chemical characteristics, (2) contamination or (3) the inclusion of solids such as sand or clay.

Quality Improvement

In his experience with rehabilitation to improve quality, the author worked on one well which tapped several aquifers at different levels. When it began to show a decided increase in hardness and salinity, a test was made to determine which water was causing the trouble. The comparative quality of the waters from the several aquifers was ascertained, using the conductivity of the waters, as determined by surveying the well with a cell, for an index. In view of the fact that the survey showed the highest conductivity to exist at the lowest perforations in the well casing, the well was filled with cement grout to cover these perforations, and the quality was materially improved.

Similar difficulties in another well, of the gravel envelope type, were traced to the higher perforations. To seal these perforations effectively, the casing was reperforated with large holes a few feet below the point where the seal was to be placed, and the gravel was removed from the envelope to the level of the new perforations. Sand was placed inside the casing until it covered 2 ft. of the new large perforations. A 12-in. diameter liner 40 ft long was placed inside the 16-in. casing, its weight causing it to sink into the sand about 18 in. Four sacks of cement were placed between the liner and the casing and allowed to set for a day, after which the space between the liner and casing was filled with cement grout. The sand was removed from the well and the gravel replaced in the envelope above the seal. This process reduced the hardness and salinity more than 50 per cent. Other wells with excess iron or manganese have been sealed in a similar manner.

Contamination due to entrance of surface water has been corrected in several wells by installing liners and seals or by using a jet to remove material from outside the well casing and sealing with cement grout.

The production of sand or clay from wells is due to: (1) improper development, (2) faulty perforations, (3) overpumping and (4) collapse of casing.

Improperly or incompletely developed wells are usually redeveloped. Whenever possible wells are developed

or redeveloped to at least 150 per cent of the capacity of the permanent well pump. When surging at this capacity fails to produce sand or clay, the wells deliver clear water and continue to do so as long as the pumping level does not fall below the level reached at completion of development.

When redeveloping fails to produce water free from sand or clay, the perforations through which sand or clay enters the well should be located and sealed by cementing between a liner and the casing or by installing a liner slightly smaller in diameter than the casing.

In several wells upon which the author has worked, the perforations were so large opposite certain strata that the coarse sand and small gravel entered through them, and there was insufficient larger gravel to form a suitable screen outside the casing. The resulting cavity was filled with clay from above when the well was overpumped.

One well suddenly decreased in production and at the same time produced clay and sand in such quantities that the water was unusable. The pump was removed and the well sounded. About 150 ft. of the well was filled, shutting off the lower perforations. The bailer encountered an obstruction at a depth of 250 ft. An impression block showed that an inner joint of the casing had opened at the seam and folded in until the opening in the casing was less than one-third the normal opening in the 14-in. diameter casing. A conventioned swage would probably enter the break outside the casing and force the damaged casing in, closing the opening entirely. Guides for a 6-in. swage were made to keep it on the inside of the broken casing. Before the swage was driven through, an impression block was lowered to make sure it was in proper position. This was repeated with 8-, 10- and 12-in. swages. The hard red steel well casing would spring back after the swage passed the break, so that after the 12-in. swage had been used the opening was approximately 10 in. A liner of 12-in. standard pipe was installed by welding a driving ring outside the pipe at the top and welding a cast-iron nose plug on the bottom. The cast-iron plug was made with a shoulder to fit the end of the pipe, with its outside diameter at the pipe the same as the outside diameter of the liner; below this it was reduced in diameter about the same as a swage. After the liner was driven to position, the cast-iron plug was broken out with the drilling tools and the pieces removed by a bailer. The well was then cleaned out and put back in operation.

Capacity Improvement

Wells decrease in capacity due to a lower water table or incrustation. To counteract decreased yield caused by incrustation or a deposition of material which clogs well screens, perforations or water-bearing formations, the following methods are used: (1) reperforation, (2) percussion, (3) chemical treatment and (4) surging.

Reperforation

The reperforation of the casing to provide new openings through which water can enter the well has been, in the author's experience, the least successful of the methods used. These failures are usually due to inaccurate logs and deterioration of the casing. About half the wells in which the casing has been reperforated have been lost entirely. The method should not be condemned because of these re-

sults, however, for with accurate well logs and good well casing, surprising results have been obtained. The capacities of several wells have been doubled by reperforating.

Use of the "Mills" knife, the "Star" perforator or even the hydraulic horizontal louver type perforator in locations where the casing had been perforated previously frequently results in tearing large holes in the casing or in the collapse of the casing.

The best results in reperforating have been obtained by using the shot perforator. The high velocity of the shot punctures the casing without the appreciable stress which results from the use of the other type of perforators. There was no evidence of damage to well casing in any well reperforated by the shot perforator.

Percussion

The shock produced by detonating either electric blasting caps (singly or in groups) or small charges of dynamite has been used to break incrustations or deposits in the perforations.

One shallow well had been drilled, cased with 16-in. 12-gage double well casing, perforated with a Mills knife, and developed to deliver 1,200 gpm. with a 20-ft. drawdown. The specific capacity of 60 was considered satisfactory for that type of well in that location.

After the well had been idle for a year, when a pump was installed to deliver 600 gpm. with a 20-ft. drawdown, it was found that the specific yield of the well had decreased from 60 to less than 4. The pump was removed and four No. 6 blasting caps detonated opposite the perforations. The pump was reinstalled, and a test showed that, with a 20-ft. drawdown, the capacity had increased from 50 to 150 gpm. To save the time and expense of removing and reinstalling the pump, guides were made to hold the caps so they would not come in contact with the casing or the pump column and shots were repeated until about 70 per cent of the original capacity was regained.

Chemical Treatment

Organic material in the perforations of the casing and in the voids of the water-bearing sands or gravel has been loosened in several wells by treating the well with a concentrated chlorine solution. The concentrated solution was placed in the wells through pipe or hose which reached to the bottom of the well. Sufficient solution was added to fill the well above the perforations. When compressed air was available, it was used to agitate the solution; in other wells dry ice was dropped into the well at 30-minute intervals. The chlorine solution was allowed to remain in the well for at least 24 hours. From 10 to 60 per cent of the lost capacity was regained in wells treated by this method when the cause of the decreased yield was clogging by organic material.

Deposits of manganese in the form of manganic salts were removed from four wells by reducing the manganic compounds to the softer, amorphous manganous compounds. This was done by treating the wells with sulfurous acid. The acid was prepared by feeding sulfur dioxide gas into a stream of water flowing through a hose into the well. This method was fairly successful. It is believed that much greater success could be obtained with this method by following the treatment immediately with swabbing or a combination of swabbing and air-lift pumping.

Calcareous deposits which had reduced well yields by as much as 70 per cent were removed by treating the wells with acid. Muriatic and nitric acids were used, but not together. Either commercial inhibitors or plain bran and molasses were always used with the acid. Compressed air or dry ice was used to agitate the acid.

Surging

Forcing water through the perforations in both directions to dislodge incrustations has been accomplished by the use of bailers, swabs, swab and air lift, or dry ice.

Rapidly raising and lowering a large heavy bailer in a well opposite the perforations alternately forces water out of and then back in through the perforations. The same effect, but with a more violent action, is accomplished with a swab.

The most successful method and the one most frequently used by the author was surging the well with a swab placed on the bottom of an air-lift pump with the swab below the point at which water entered the air lift. The advantages of this system of agitation and cleaning are that: (1) a differential head can be maintained at the swab and high velocities obtained through the perforations, (2) most of the solid material entering the well is immediately removed, (3) the results of the swabbing are immediately observable when the material discharged by the air-lift pump is sampled and (4) the time of swabbing and cleaning out wells was materially decreased.

This method has been used by the author on more than 30 wells, increasing the yield of each. The application of this system was not limited to the rehabilitation of old wells, but was also used to develop new wells.

One well was drilled, cased and perforated with a hydraulic, horizontal, louver type perforator. The well was originally developed by surging with a bailer and swab and a large capacity pump. As the yield of the well was below expectations, the pump was removed and the well again swabbed, the pump reinstalled and a drawdown test made of the well. With a drawdown of 100 ft., the yield was only 300 gpm., giving a specific capacity of only 3. The air-lift swab was used to develop the well further. This additional development increased the yield to 800 gpm. with a drawdown of 60 ft., or a specific capacity of over 13.

In rehabilitating old wells, improperly developed strata were often found, and better gravel screens were developed.

If liners or other obstructions did not permit the use of the air-lift swab, dry ice was used. A plate was welded to the top of the casing, to which a 4-in. pipe coupling was welded, the plate inside the coupling being cut out. A 4-in. gate valve was then installed on the coupling. A small opening was cut in another portion of the plate, and a pressure gage attached. Dry ice sticks, $3 \times 3 \times 10$ in. in size, were dropped into the well through the open gate valve, which was immediately closed. Pressure developed by the change of the carbon dioxide from the solid to the gaseous state forced the water out through the perforations. As soon as the pressure gage showed a drop in pressure, the gate valve was opened as rapidly as possible, and water from outside the casing rushed back through the perforations. This procedure was repeated until the pressure shown by the gage reached the same maximum recorded by the previous charge.

Conclusion

Before deciding upon a method of well rehabilitation, it is well to obtain all the information available about the well and, if possible, to determine the material or materials which may be clogging the perforations or screen. Someone with imagination and experience should be selected and given full responsibility for carrying out the program. The success of any method requiring the use of such well drilling equipment as, for example, the air-lift swab, depends as much on the skill of the person operating the equipment as on the engineering.

Radio Communication for Water Works

By Raymond G. Ridgely

A contribution to the Journal by Raymond G. Ridgely, Director, Municipal Water Works, St. Petersburg, Fla.

A NUMBER of years ago, the city of St. Petersburg, Fla., equipped some police cruisers with radio communication sets and installed a central short-wave broadcasting station. A few years later, the Water Dept. decided to install receivers on service trucks, to be controlled by the police band. The value of instant communication to the field crews (who could not, however, call the central station), was very quickly demonstrated.

In 1930, the Pinellas Water Co., which supplied St. Petersburg with water, constructed a supply system and works 35 miles from the city. Telephonic communication with this project soon proved difficult and uncertain, and efforts were made to secure permission from the Federal Communications Commission to install some type of radio communication. At that time, however, the commission refused. Dr. Gerhard Fisher of the Fisher Research Lab., Palo Alto, Calif., was then engaged to work out a solution, and began working on a directionalized wave to be impressed upon the water main connecting the source of supply with the city plant.

The war, of course, disrupted all this work. In December 1940, however, the city had acquired the properties of the company, and the application for a radio license was renewed. This time the application was approved; the city was licensed to operate three permanent stations and one mobile unit utilizing frequency modulation on a wave length of 39.860 mc. When the ultimate channel to be used is definitely assigned, it is intended to install two-way systems in the trucks.

For a system characterized by widely separated plants and a distribution system covering 58 square miles, radio communication has become a necessary adjunct to operations. The results at St. Petersburg have been such as to make the operators enthusiastic and to convince them that many plants would benefit from similar operations.

Abstracts of Water Works Literature

Key: In the reference to the publication in which the abstracted article appears, **34**: 412 (Mar. '42) indicates volume 34, page 412, issue dated March 1942. If the publication is paged by the issue, **34**: 3: 56 (Mar. '42) indicates volume 34, number 3, page 56, issue dated March 1942. Initials following an abstract indicate reproduction, by permission, from periodicals, as follows: *B.H.—Bulletin of Hygiene (British); C.A.—Chemical Abstracts; P.H.E.A.—Public Health Engineering Abstracts; W.P.R.—Water Pollution Research (British); I.M.—Institute of Metals (British).*

HYDRAULICS

The Venturi Flume Meter. A. Linford. Wtr. & Wtr. Eng. (Br.), **50**:19 (Jan.'47). Venturi flume meter does not suffer from head loss nor upstream silting as does weir. Inglis and Crump were first to use this device in irrig. work. Clements Herschel referred to Venturi flume as "Venturi tube with lid off." Meter can usually be designed so that only necessary to measure upstream depth of flow. In throat at each flow rate, depth of water maintains itself at certain critical value at which water has min. energy content. Can be proven that:

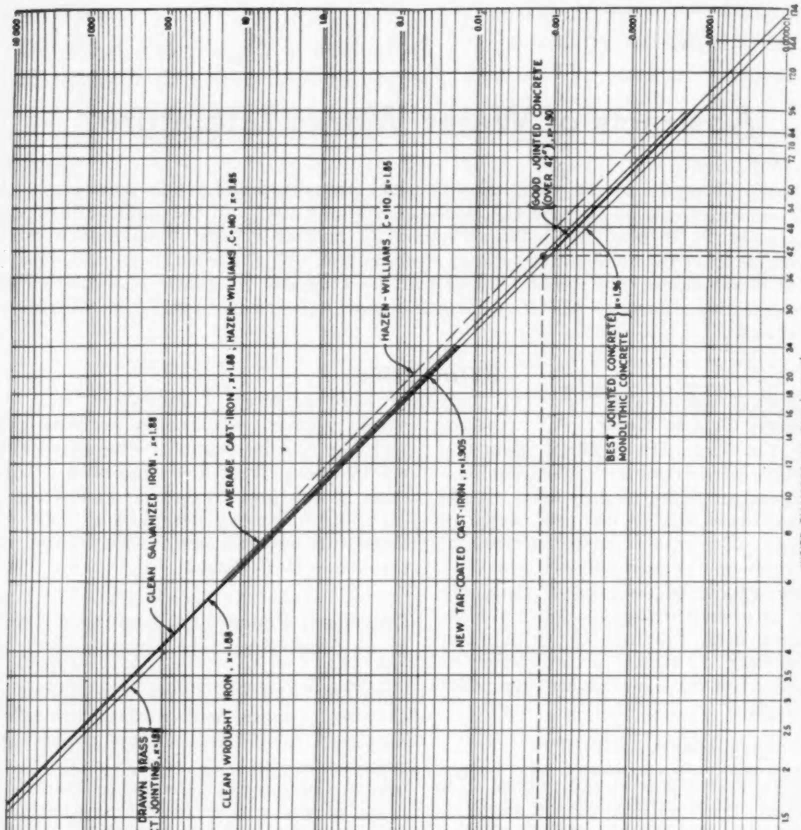
$$Q = 0.385\ \sqrt{2_g}CW_2H_1^{3/2}$$

in which Q is flow, g is accel. due to grav., C is dischg. coeff., W_2 is width at throat, and H_1 is depth in approach channel. Where velocity of approach is allowed for:

$$Q = \sqrt{2_g}CWH_1^{3/2}\sqrt{\frac{(1-x)E^2x^2}{E^2 - x^2}}$$

in which E is W_1/W_2, x is H_2/H_1, and W_1 is width of approach channel. For rectangular flumes, approx. formula becomes $Q = 2.935\ W_2H_1^{3/2}$ and the more accurate formula becomes $Q = 7.62\ W_2H_1^{3/2}\sqrt{\frac{(1 - x)E^2x^2}{E^2 - x^2}}$ where Q is in cfs. and other dimensions are in ft. Where E is greater than 3, agreement between eqs. becomes closer than 2%. At low flows, C tends to decrease. There is head loss in flume and 100% head recovery cannot be obtained. Actual value of H_3 (downstream depth to assure free flow) may be less than limiting value due to head loss in standing wave, and this form of meter frequently referred to as "standing wave flume." To maintain "free" flow conditions, essential to obtain approx. relationship between depth and flow rate in channel. Frequently flume will be under "free" flow conditions at higher rates of flow and "drowned" at lower rates. To avoid increasing head loss at max. flow, flume can be constructed with bottom contraction.—*H. E. Babbitt.*

Centrifugal Pumps—An Alternative Theory. H. H. Anderson. The Engineer (Br.), **182**: 106 (Aug. 2, '46). Momentum theory of centrifugal pumps given by Euler's eq. ($h = uw/g$) questioned by designers as it is alleged to give heads almost double those obtained in practice. R. L. Daugherty, and others, suggested difference between rel. angle of water flow and of impeller blade. Theory put forward in this paper based on assertions: (1) that difference of angle can be deduced from theoretical considerations alone, by equating impact on impeller blade to whirl produced and (2) that head due to centrifugal force must be added to head due to impact. Assumed that inlet to impeller is radial and that following areas, and velocities, are equal: suction branch, inlet radial impeller area, outlet radial impeller area, delivery branch. Nomenclature used is: u is peripheral veloc. of impeller in fps., F is radial veloc. of water in *fps.*, Q_f and Q_F are quantity flowing in cfs., β is angle of impeller blade to tangent at outlet; α is angle of volute or absolute dischg. to tan at outlet, A_f is radial inlet area of impeller, A_f is radial outlet area of impeller; A_v is area of volute throat, R_1 is inlet radius of impeller, R_2 is outlet radius of impeller, k is $1 - (R_1/R_2)^2$. Exisiting theory assumes that water leaves impeller periphery with relative angle equal to outlet angle of impeller, and that whirl equals $u - F \cot \beta$. At



Courtesy Boston Society of Civil Engineers

FIG. 1. Covil Chart for Pipe Selection

quantity Q_F, head will be zero, since there can be no impact between blade and water; therefore no torque. Proposed theory shows how head, up to Q_f, quantity for max. eff., is roughly ½ value given in existing theory and demonstrates how volute area A_v dets. max. hydraulic eff. pt., Q_f, beyond which head falls steeply to zero at Q_F. Head and quantity characteristics of volute pumps have been calcd. and compared graphically with tests. Turbine pump shows complete agreement between calcns. and observations, although they seem too high. Theory presented involves most elementary hydraulics, fits facts reasonably and is submitted as complete answer to difficulties that have caused designers to disregard theory in past. Theory will apply equally well to fans.—*H. E. Babbitt.*

A Practical Formula for the Flow of Water in Pipes. WM. F. COVIL, J. Boston Soc. Civ. Engrs. **32**:1:18 (Jan. '45). Complicated formulas reduce to $h_f = k_pQ^x$, where h_f is head loss per 1000′ of pipe, k_p is friction characteristic, Q is quant. in cfs. Term k_p varies with diam. but is const. for given diam. and unaffected by quant. Exponent x varies with type and condition of pipe, but for same type of new pipe is unaffected by diam. Darcy formula $h_f = \frac{fL}{D}\frac{V^2}{2g}$ fell into disuse because f varied with type of pipe, diam., veloc. and hence quant. Exponential formulas came into use. Reynolds no. caused reconsideration although usually temp. taken at 55°F., so Reynolds no. may be omitted. Darcy formula may be written $h_f = kfQ^2$ also $R_e = \frac{4Q}{\pi D\nu}$. For const. temp. $f \neq k_1Q^{-a}$ or $f \neq \frac{k_1}{Q^a}$. Logarithmic plotting of experiments verifies this last eq. k_1 should be measure of skin friction; is a dimensionless coef. varying approximately constantly with diam. and increasing with roughness. By substitution $h_f = kk_1Q^x$ where x is 2-a. Log-log plotting gives values of a between 0.15 and 0.07; x between 1.85 and 1.93; k_1 = 0.018 to 0.040; k_1 varies about as first power of diam. Comparing Hazen-Williams, Scobey, Manning and Darcy formulas, former seems to approx. widest range best. Minor losses may be added by using separate k for each bend, valve or other irregularity. k_p and x are valuable only for reasonably straight pipe. Allowance for bend and valve losses important. Value of x may be obtained by log-log plotting. If result is more than 2, minor losses have not been subtracted. $k_f = k_pL$ and $k_f = \frac{H_f}{Q^x}$. From test results these may be found and k_1 determined. k_1 increases with diam. and should plot as straight line on log or semi-log paper. x may increase with size of pipe but is const. for one diam. Good inf. lacking about large sizes of pipe but results tabulated show x between 1.88 and 2.03, k_1 between 0.0143 and 0.0218. Age of pipe data not sufficient. Temp. may be neglected for normal water supply studies. Alignment chart (Fig. 1) permits quick solution of problems as to size, flow and head loss. Author wishes more data on large pipes and believes simple practical formula has been made available.—*Charles H. Capen.*

Hydraulic Properties of Asbestos Cement Pipe. T. D. LEECH. New Zealand Eng. (N. Z.) pp. 114, 206 (May 10 & June 10, '46). Exptl. investigation by staff of Auckland School of Eng. into properties of Fibrolite asbestos cement pipes described and results given. For flows up to critical values of Reynolds' number, pipes hydraulically smooth; beyond those values they behave as rough pipes. Purpose of study was to det. loss of head for given unit length, discharge being varied over practical ranges. Results had to be presented in form showing loss of head through 100′ of pipe for typical discharges. Research now recognizes 3 quite different pipes—streamline, transitional and turbulent, depending on velocity. Up to critical velocity (inducing change in flow) loss of head proportional to velocity in streamline flow at low velocities, then at higher rates of flow, steady motion becomes sinuous and turbulent and water particles may run perpendicular to general direction of flow, and marked changes occur in relationship between loss of head per unit length and mean velocity of flow. Sizes of Fibrolite pipes and range of normal flows gave rise to conditions of turbulent flow. Equations in streamline study given; for any set of conditions, namely, given fluid flowing through given pipe, V (mean velocity of flow), g (acceleration by gravity) and m (hydraulic mean depth) are constant, therefore loss of head proportional. For turbulent flow, principle of dynamic similarity can be used to deduce a satisfactory law governing flow; expression given. Quant. $\frac{V.d}{v}$ often called Reynolds' number for the

pipe. App. used in tests described and pipe data obtained with formulae worked out given. Tables of results show basic hydraulic properties of nominal 4″ Fibrolite pipes and friction in 4, 6 and 9″ pipes. Second installment of paper contains comparison of work with Williams and Hazen eq. (of empirical origin) for computing velocity of flow in pipes; eq. given and values for C (allegedly a constant for any one pipe) tabulated. Appendix I works out kinematic viscosity of water and air, and Appendix II deals with boundary layer theory in respect of rough pipes, and quant. anal. included; hydraulically, smooth or rough pipes depend on rugosities on inner surface, these interfering with laminar sub-layer flow.—*Ed.*

National Research Council's Hydraulic Laboratories, Ottawa, Canada. ANON. Wtr. and Wtr. Eng. (Br.), **49**:122 (March '46). At laboratories near Ottawa scale tests are carried out on models of sluiceways, dams, and other parts of hydraulic installations, harbors and vessels. 60-ft. model of section of Fraser River, B. C., is under construction in laboratory. In operation in laboratory is model of Kingsville Harbor on Lake Erie for use with wave-making machine. 67′ glass-sided flume runs almost length of lab. Stop-log model used to measure direction and magnitude of forces acting on stop logs of emergency dams. Addnl. buildings planned. —*H. E. Babbitt.*

BOILERS AND FEED WATER

Developments in the Science of Water Treatment. W. F. GERRARD. Steam Engr. **15**: 137, 187, 209, 222, 258, 266, 279, 309, 359, 368 ('46). Review of problems of modern treatment of boiler feed water. Problem largely consists in elimg. O, CO_2, $Ca(HCO_3)_2$, $Mg(HCO_3)_2$, $CaSO_4$, $MgSO_4$, $MgCl_2$ and SiO_2, or in counteracting their effects. D. O. and CO_2 chiefly responsible for corrosive action on metals, and can be elimd. by mech. de-aeration followed by treatment with Na_2SO_3 to remove residual O. pH value should never fall below 9.0. Corrosion can be inhibited by cathodic process, by treatment with Na chromate, or by use of colloidal substances such as tannin, starch and Na_2SiO_3. Protective coatings such as bitumen, paint, plating and graphite can be used to protect metal. Principles of softening water by zeolites discussed. Disadvantage suffered by zeolites in treating water for steam generation is formation of $NaHCO_3$, which is decomposed by heat to form CO_2, Na_2CO_3 and NaOH. Double-exchange system, phosphate, apatite and colloidal conditioning, and lime-soda process of softening discussed. Foaming may be caused by soapy substances, oil in alk. water, alkalis in multitubular smoke-tube boilers, org. matter such as humus and sludge from sewage works. For caustic embrittlement 3 main conditions fundamental: caustic alkali must be present in boiler water; steel must be stressed beyond its elastic limit; and temp. must exceed some value between 185 and 212°F.—*C.A.*

Treating Waters, Especially Boiler Waters, and Composition of Same. WAYNE L. DENMAN. U.S. 2,400,543 (May 21, '46). Compn. and procedure for waters, and in particular boiler waters, to prevent foaming described. Compds. effective in treatment belong to satd. and unsatd. aliphatic amines contg. 11 or more C atoms and being substantially sol. in H_2O and showing little tendency to distn. Boiler waters having excess alky. have tendency to foam and treatment described inhibits foaming. Preferred form of antifoam compn. that of gelatinized product. As illustration, such mixt. may be made by combining an amine, as heptadecylamine 4, tannin ext. 94 and corn meal 2%. 5 mixts. given in table and amine content may vary from 1–35%. 4 tables included further to describe variations of amine, tannin ext., bentonite and corn meal. British gum may be substituted for corn meal. Starches or dextrins may also be employed to give desired viscosity. Discussion of period of foaming inhibition and quants. of mixt. employed sets forth claims for treatment.—*C.A.*

Methods for Taking Water Samples. R. M. STIMMEL ET AL. Am. Ry. Eng. Assn. Bul. *No. 455:* 83 (Nov. '45). Where special sampling equip. not available for obtaining boiler water samples, water column or water glass drain can be used by taking care to flush thoroughly and by drawing off sample slowly and carefully in order to avoid loss from flash.—*R. C. Bardwell.*

Once-Through and Recirculating Cooling Water Studies. H. Lewis Kahler. Proc. Ann. Water Conf., Eng. Soc. Western Pa., 5: 39 ('44). Scale formation from cooling water studied in lab. app. in which wt. of scale deposited on metallic heating surface detd. With once-through cooling, amt. of scale formed increased with increase of: H_2O temp., heat transfer and skin temp., and concn. of scaling compds. Langelier's index related qualitatively to deg. of scale formation in untreated waters. Best treatment obtained with mixt. of complex phosphate and org. materials of tannin type. Pptd. $CaCO_3$ crystals distorted by surface-active agents. In recirculating systems, $CaCO_3$ formation increased as cycles of concn. increased.—*C.A.*

Single- and Two-Stage Phosphate Hot-Process Softeners. C. E. Joos. Southern Power & Ind. **64**: 9:68 ('46). For relatively soft H_2O having hardness not exceeding approx. 50 ppm., good practice to utilize phosphate as primary treating reagent instead of lime and soda ash. With use of lime and soda ash permanent hardness will vary inversely in proportion to excess Na_2CO_3 carried, and good practice would indicate that max. reduction of hardness would be in order of 10 ppm. With use of phosphate, on other hand, zero hardness by soap test regularly obtained, equiv. to hardness usually less than ½ ppm. Under these conditions boiler conc. crystal clear and sludge accumulation in boiler absent or negligible. Diagrams given of single- and two-stage softeners.—*C.A.*

Operating Experience With the Wirbos-Permutit Treatment of Boiler Water. E. Schmitt. Z. Ver. deut. Ing. (Ger.) **88**: 395 ('44). $Ca(OH)_2$ added to raw water and pptd. $CaCO_3$ removed in reactor. This reactor charged with sand on which $CaCO_3$ accumulates. Original sand had avg. diam. of 0.2 mm. This diam. increased to 1.5 mm. through deposition of $CaCO_3$ without decreasing effectiveness of reactor. With particle sizes over 2.0 mm., increased turbidity in effluent from reactor observed. Water passed from reactor after detention period of 10–15 min. to gravel filter, then through Na zeolite exchanger. After treatment with Na_3PO_4 (6 ppm.), water degasified and admitted to boiler. Operation expense 43% of that for conventional lime-soda softener. Residual hardness 0.5 ppm. as compared to 21 ppm. residual hardness for lime-soda softener.—*C.A.*

Embrittlement Cracking in Waters Containing Potassium Salts. A. A. Berk & N. E. Rogers. Trans. A.S.M.E. **67**: 329 ('45). Embrittlement detector used to test dil. solns. in range of concns. encountered in boiler operation. Effect of KOH solns. on stressed steel also detd. with concd. solns. in tension testing device. KOH solns. produced intercryst. cracking in stressed steel specimens both in embrittlement detector and in tension tests, and at 250° small amts. of SiO_2 greatly accelerated attack. KNO_3 and quebracho ext. tests show that practical embrittlement inhibitors in solns. contg. NaOH alky. also effective in KOH solns. K_3PO_4 apparently can be used as effectively as now widely used "co-ordinated" Na phosphate treatment. K metasilicate and K disilicate also caused no cracking.—*C.A.*

Silica in Steam—Its Causes and Prevention. Eng. & Boiler House Rev. (Br.), **60**: 34, 48 (Feb. '46); Corr. & Mat. Prot. (Br.), **3**: 4:24 (Apr. '46). This is second part of article dealing with operating experience in h-p., high capac. boilers of natural and forced-circulation types. Use of potassium salts instead of sodium for boiler water conditioning was made in endeavor to prevent formation of silica scale. First trouble that could be attributed to potassium was failure of hand hole. Monel-clad asbestos cap gaskets. Due to potassium, amt. of iron oxide in boiler deposits also increased. Also due to increase of iron oxide, caustic attack of boiler iron increased. Chlorine added to dil. caustic concn.—*Ed.*

Chemical Cleaning Takes the Bull Work out of Scale Removal. E. W. Feller & Guy F. Williams. Power. **90**: 9: 74 ('46). When solvents can be used, scale easily removed. Comparison of HCl and H_2SO_4 indicates that from standpoint of rapidity of action and qual. of by-products HCl better. Inhibitors tabulated and discussed. Various elements and compds. cited that tend to corrode even in presence of inhibitors. Dangers from gases produced suggested and there is general discussion of expediting and wetting agents for speeding up solvent action and getting uniform and speedy contact and penetration.—*C.A.*

Feed Water Treatment for Locomotive Use. T. W. Hislop. Mech. Eng. **67**: 515 ('45). As result of prelim. tests, treatment of boiler feed water extended to entire New York

Central system, and at present time 253 water supplies treated; 34 lime-soda softeners, 78 liquid proportioners and 141 by-pass feeders employed. Boiler-washout periods have been extended in most cases from 6 to 30 days and mileages between shippings have increased over 100%. Waters with high hardness, or waters of relatively low hardness but large consumption, treated by lime-soda softener, usually in charge of full-time operator, but chem. addns. to other plants made by men available at location. Chem. anals. of feed water and of boiler water used to ckeck up on treatment, and water service inspectors make regular visits to plants and terminals. Automatic, continuous boiler blowdowns supplemented by manual, cab-operated blowdown when necessary.—*C.A.*

Methods of Removing Oily Deposits From Inside of Locomotive Boilers. J. J. Dwyer et. al. Am. Ry. Eng. Assn. Bul. *No. 455:* (Nov. '45). Where open type feed water heaters used, that portion of cylinder oil in exhaust steam used for heating water goes directly into boiler with feed water and has caused trouble in many cases. Test of chem. means for removing these oily deposits indicated that boiling with neither soda ash solns. from 0.1 to 0.5 lb./gal. nor with caustic soda at 0.15 lb./gal. was satisfactory. Mixt. of 250 lb. Na_3PO_4, 100 lb. sodium metasilicate, 20 lb. NaOH and 30 lb. wetting agent boiled for 20 hr. in 6000-gal. capac. boiler was helpful but not entirely satisfactory. Best results obtained by boiling at 100-lb. pressure for 20 hr., using sodium metasilicate and wetting agent in proportions 0.5 oz. and 0.05 oz., respectively, to 1 gal. of water. Such boiling required in some cases every 30 days.—*R. C. Bardwell.*

What Water Treatment Has Done for the Railroads and What Is To Be Done to Further Improve Locomotive Boilers. John F. Powers et. al. Proc. Master Boiler Makers Assn. p. 60 ('45). (1) Made possible long runs of steam locomotives; (2) permitted use of high power locomotives; (3) reduced fuel consumption; (4) saves railroads not less than **$30,000,000** annually; (5) during each war year saved substantial tonnage of steel; (6) contributed very largely to availability of steam locomotives, enabling 42,000 locomotives to deliver nearly twice work done by 64,000 locomotives in World War I; (7) reduced shopping time of locomotives to min. required by mach. repairs; (8) contributed to performance record where boiler failures virtually elimd.—*R. C. Bardwell.*

New Developments in Water Conditioning for Diesel Locomotive Radiators and Flash Boilers. M. A. Hanson et al. Amer. Ry. Eng. Assn. 48, Bul. No. 462: 203 (Nov. 1946). Problem of corrosion prevention throughout entire cooling system of diesel locomotive quite complex, due to number (about 9) of different metals encountered. Accumulation of scale and sludge must be avoided and corrosion prevented. Experience indicated that treatment for scale elimn. or use of demineralized water and corrosion inhibitors, usually chromates, necessary. Scale and corrosion has caused short life of tubes in many of the Clarkson type steam generators. Acid washing not entirely successful in removing scale deposits, particularly those of silica type. Tests being conducted, using demineralized waters with pH control and organic oxygen absorbers. Further research and investigation considered necessary. Photographs showing extensive list of troubles encountered included in report.—*R. C. Bardwell.*

Experience With Potassium Treatment at Windsor Station. W. L. Webb. Trans. A.S.M.E. **67**: 325 ('45). With hardness, Al_2O_3 and SiO_2 in quant. entering cycle through condenser leakage, wall tube losses in two 1350-psi. boilers extensive under conventional Na treatment. When both boilers operated under K treatment, with mE/l. ratios of K/Na = 2 and SiO_2/OH = 0.5, treatment did not prevent failure of wall tubes on which analcite had already formed. Deposit rates in 30,000-kw. low-pressure turbines receiving steam from these boilers substantially same as under Na treatment. Two boilers then chemically cleaned and one put on K treatment and other on Na treatment. Both boilers receive same feed water. Test period of 7 mo. too short to permit drawing conclusions, except that sludge in K-treated boiler less adherent than in Na-treated boiler. No hard scale deposits observed in either.—*C.A.*

History of Potassium Boiler-Water Treatment at Springdale. L. E. Hankinson & M. D. Baker. Trans. A.S.M.E. **67**: 317 ('45). K salts substituted for Na salts for water conditioning for both high- and low-pressure boilers. Purpose was to prevent formation of SiO_2 scale in boiler and deposition of SiO_2 on turbine blades. Na compds.

formerly used flake NaOH, anhyd. Na_2HPO_4, and Santosite (alk. Na_2SO_3). K compds. substituted $K_4P_2O_7$, KOH and K_2SO_3. Sulfite addn. later discontinued. With Na treatment, gasket failures largely self-sealing. With introduction of K, previously established seals dissolved. After re-gasketing and getting all tubes properly rolled, this type of trouble practically disappeared. K-conditioned water will not seal even smallest leak. When K salts used, all surfaces where sludge does not accumulate clean. With Na treatment, film of SiO_2 scale often present. K silicate will not deposit as scale but K Al silicate and K Fe silicate will form under sludge deposits where boiler water can conc. Indications are that amt. of scale not as heavy with K as with Na, and that K scales will disintegrate in HCl. When mE/l. ratio K/Na less than 8 to 1, condition of interior boiler surfaces similar to that found when Na treatment used.—*C.A.*

DISTRIBUTION SYSTEMS—CONSTRUCTION AND MAINTENANCE

The Bacterial Deterioration of Water in Distribution Systems. R. FERRAMOLA. Rev. Administración Nacional del Agua. (Argentina) 10: 106, 266 (Apr. '46). Bact. deterioration of water in South American distr. systems owing to infection from jute jointing material discussed, and expts. carried out to find better substitute for yarn. It might appear from hygienic point of view that little importance need be attached to coliform bacteria appearing in distr. system when traceable to yarn, but we are not yet in position to establish by bact. differentiation origin of coliform organisms found in sample of water, nor, for that matter, are we certain of deg. of signif. to be attached to each type. In addn., jute yarn is capable of supporting growth of pathogenic organisms such as *Salmonella typhi*. A medium, made up of macerated jute in tap water, was sterilized and then inoculated with pure cultures of *Esch. coli*, *Aerobacter aerogenes*, *Salmonella typhi*, and *Salmonella paratyphi A*. Growth of bacteria was abundant. Search made for substitutes for jute, bearing in mind that substitute: (*a*) should not act as culture medium for bacteria; (*b*) should not contain substances which would render water unpalatable or unsafe to drink; (*c*) should be cheap and readily obtainable; (*d*) should be easy to apply as jointing material. After considering number of materials, decided to investigate glass wool, asbestos, rubber and paper (impregnating jute with bactericidal agents not considered practicable). These materials tested against organisms mentioned above. Results with rubber very satisfactory; it appeared to contain inhibitory substances. Unfortunately rubber not available for civilian requirements, and further tests were abandoned. Material contg. asbestos also gave satisfactory results, but paper material produced slight multiplication of *Esch. coli* and *Aer. aerogenes;* pathogenic enteric organisms tested died out rapidly. Trials were next carried out on exptl. mains, conditions in distr. system being imitated as closely as possible. Unsatisfactory nature of jute again observed; little difference in behavior of other substances tested (glass wool, asbestos, rubber and paper) in all of which no bact. growth took place. Decided, taking all matters into consideration, to try out paper in actual distr. system, although other materials under test superior from bact. point of view. Workmen had no difficulty in manipulating paper, which was of newsprint texture twisted into pieces of suitable thickness. Mains contg. paper have been under continuous observation for 10 months, samples being drawn at weekly intervals and, on one occasion, water in main artificially contamd. with culture of *Aer. aerogenes*. Such an infection in jute-packed main would have persisted for long time, but this infection disappeared within a few days. Paper proved satisfactory substitute for yarn; any growth-promoting elements in paper diffuse into water and are quickly washed away in flow.—*B.H.*

An Old Specification for C.I. Pipe Joints. A Solid Lead Joint. W. S. OSMAN. Wtr. & Wtr. Eng. (Br.) **49**: 457 (Aug. '46). Joint filled with lead, no hemp or yarn being used. Joints were used in Southampton and some miles were used on Poole Water Supply system. Following are abstracts from specifications: Each joint shall be made by inserting in joint space 3 complete rings of

¼-in. diam. lead wire and calking tightly into bottom of socket. Joint then run full with molten lead. Each joint shall be properly chiseled, trimmed off and set up or calked tightly.—*H. E. Babbitt.*

Reinforced Concrete Spun Pipes in Argentina. Mario Ninci. The Engr. (Br.) **180**: 501 (Dec. 21, '45). Aqueduct between Rio Tercero and San Francisco, in Province of Cordoba, is interesting public work. Object is good water supply for San Francisco and 11 other places on Santa Fé Ry. Works comprise: (a) intake works, with filtering gallery, (b) aqueduct, consisting of reinforced concrete pipe varying in diam. from 50 to 60 cm., and (c) purif., storage and distr. works in San Francisco. Total length is 167 km.; capac. amtg. to 200 l./sec.; flow being entirely by gravity under total head of 105 m. Estd. cost was 7 million Argentine pesos. Pipe used on greater portion has following characteristics: internal diam. of 600 mm.; wall thickness of 62 mm.; length of 3.50 m.; working pressure equal to 12-m. water column; resistance to external pressure 2000 kg. per lin.m. of pipe. Concrete mix is 500 kg. cement per cu.m. of concrete. Reinforcement consists of 2 spirals of 6-mm. diam. round steel, spaced at 9-cm. distance from one another, and 40 longitudinal bars of same diam., all joints between longitudinal and spiral reinforcement being electrically welded. Centrifugal method of making pipes adopted. Characteristics of spun concrete are: (a) well-compacted structure free from porosity and voids; (b) uniform distr. of constituent materials; (c) the concrete may be mixed in semi-dry state, thus obtaining mixture of adequate plasticity without excess water, and during rotation all excess water expelled; (d) by above properties of high density and low water/cement ratio pipe can satisfy requirement of min. absorption, high resistance and complete impermeability. Procedure giving best results was to apply 2 successive layers to depth of 42 mm., speed of rotation of mold being 260 rpm. for 17 min. Machine stopped and excess water removed, after which started again and "compactor" introduced. This device consisted of 2 flexible steel sheets fixed to end of loading skip. Its effect was to smooth interior of pipe. After this operation third layer of 20 mm. of concrete added, mold spun 5 min., excess water removed and compactor passed in and out to give final finish to inside of pipe. After completion, impossible to damage pipe with steel tool. After lapse of 3 hr. from time of mfg. of last pipe in forenoon, curing process begun in heating chambers. Steam at 45°C. passed through them for 12 hr. Cured pipes stood for 30 days before being pressure-tested. Tests of resistance to external load and to absorption made by current methods. Test consisted of submitting pipe to water pressure of 10-m. column for 14 min. and 19-m. column for 16 min. Mfr. of pipes started May 29, '41, gave unsatisfactory impermeability tests after 30 days. Reasons for defects were: (1) fissures and leaks which appeared in adjacent zones at ends of pipes and tendency towards concn. of leaks at joints of molds; (2) seepage, weeping and dispersed fissures at various points over surface of pipe. To obviate fissures, proposed that pipes should be cured by immersion in water tanks. Observation of pipes in certain successive tests when concrete was impregnated with water indicates that pipes had been prematurely denied required dampness. Complete curing important element in production of impermeable concrete. Shrinkage due to fall in temp. when removing pipes from steam cylinders inappreciable. Calcn. gives avg. tension in concrete (due to shrinkage in steel) as 5.7 kg. per sq.cm. If shrinkage reaches these values when concrete has not reached resistance corresponding to age of 7 days, tension will result in formation of fine fissures within pipe wall. This explanation justifies proposal to cure pipes by total immersion. Results of curing tests give best solution: tank curing cheaper and allows reduction of steam curing from 12 to 6 hr., insuring max. use of molds, and elims. need for moistening, which requires careful attention. Following deductions can be made: (1) with any cement which corresponds to official specification of *Obras Sanitarias de la Nación*, concrete spun pipe of excellent qual. can be made. (2) Curing by total immersion for 5 days, after short steam treatment, gives good results, and is economical because it allows early dismantling of mold. (3) Once mfr. has been harmonized with erection of pipes in finished work, they can be filled with water immediately until time arrives for pressure testing. (4) When taking into acct. calcn. of tension which affects impermeability, when pipes being designed to resist high pressures, test pressures reduced and wall thickness adopted which diminishes percentage of steel, thus permitting more favorable distr. of tension and increase in working stress of steel.—*H. E. Babbitt.*

Galesburg's Concrete Water Tower. Am. City. **60**: 106 (Nov. '45). Army's Mayo Gen. Hospital was located in Galesburg, Ill., in spring '43. City agreed to furnish water for hospital, which necessitated increasing supply and enlarging distr. system in northern section of city. Previously supply obtained from wells with 2000-gpm. capac. which pumped into 2 open concrete ground-level storage tanks. Pressure in distr. system maintd. by direct pumpage. Supply increased by drilling another well 2400′ deep into same strata as city's other 2 wells. Pump having capac. of 600 gpm. placed on this well, and new feeder main laid into area where hospital was to be located. A 0.75-mil.gal. concrete water tower constructed in area to provide necessary elevated storage. This tank made of prestressed concrete and 60′ in diam. with water depth of 35′. Its constr. required first building of concrete shell 11″ thick, on which were placed circumferential steel bands stressed to predetermined figure by means of turnbuckles. Concrete 4½″ thick then placed on outside of bands. Tank rests on 3 concentric rings of concrete 80′ 6″ high. Diam. of outside ring 62′ 7″, intermediate 42′, and inner ring 21′ 4″. Each cylinder 8″ thick. Rings capped with a 2-layer section of concrete which serves as bottom of tank. Bottom layer is 5½″ thick, top layer is 24″ thick, and poured with construction joints over supporting rings. Booster pumps and chlorinator placed underneath tank and inside center supporting ring.—*P.H.E.A.*

Winter Pipe Laying at Peterborough. R. L. Dobbin. Eng. Cont. Rec. **59**: 3: 74 (Mar. '46). To maintain existing organization and aid in reducing accumulated demand for extensions, main laying continued throughout winter, warming bells and spigots prior to pouring joints. Mains allowed to remain empty until spring to avoid increased danger of freezing due to frozen backfill. Extra cost justified by progress made.—*R. E. Thompson.*

Winter Precautions at Regina, Saskatchewan. J. W. D. Farrell. W.W. Eng. **98**: 1501 (Dec. 26 '45). Regina, pop. 60,000, has periods during winter of subzero temp. lasting from few days to 3 wks. Water supply from wells; generally above 40°F. Freezing of mains prevented by 7½′ cover. Unavoidably exposed pipe protected with 2″ of asbestos covering. Hydrants inspected before onset of severe weather and drained or pumped out. Packing gland satd. with antifreeze and cap thread oiled. Routine inspections made through winter as time permits. Frozen service connections rare; thawed out by plumbers using small steamer, or electrically by City Light & Power Dept. at $10 per connection. Small water meters have breakable frost bottom and householder charged for repair if responsible. Warnings sent to premises where meter frozen previous winter. System includes 3 reservoirs, most exposed one being of 5 mil.gal. capac., about ½ below ground, with no earth banked on sides or over roof. This reservoir still in service after 30 winters, though showing some damage from ice. Leaks located either by heaves on ground or by drilling test holes. When backfilling excavated earth, unfrozen earth from center of pile packed around pipe. When necessary sand and gravel mixed with remaining frozen lumpy material, which is then tamped in place.—*P.H.E.A.*

Effect of the Grid on Water Works Pumping Practice. H. R. Lupton. Surveyor (Br.), **104**:819 (Dec. 28, '45). Preferable to utilize distributed sources and elim. initial cost and continuous friction losses inseparable from wide-flung reticulation. Centralized pumping may involve increment of lift, due to depression of water table assocd. with concd. pumping, or, in river supply, to necessity of placing intake at level low enough to insure sufficient vol. Multiple small stations not more subject to breakdown than few large stations with stand-by plant if there are sufficient inter-connection. Multiple small communities may often be satisfied with sufficient constancy with min. of "reservoirage." Availability of electricity to drive small pumping stations may obviate great expenditures on mains and storage. Argument may be overridden by purif. of water. Treatment at small stations unlikely to present insuperable obstacle, and dispersion of supply may well increase in future. Elec. power begetter of small repumping station and booster. Repumping station, by avoiding necessity of raising whole supply needed by local "pimples," may save considerable power. Provision of multiple service reservoirs to suit each zone uneconomic. Chief advantage from boosting plant is cheapening of new trunk mains, fuller use of existing ones and postponement of their duplication. Existence of grid has prevented many long-continued breakdowns in elec. supply. Nevertheless, any pumping station must be pro-

vided with reliable and quickly started stand-by. If steam-driven, necessary to keep boilers banked, involving cost in coal and labor. Where diesel engines are stand-by, preferable to drive alternators (generators) to provide alternative elec. supply. In river supplies, cheapness of electrically driven pump will allow installation of more units than is possible where each unit is provided with prime mover. Promptness with which electrically operated pumps can be started renders them suitable for taking advantage of sudden flushes. Growth of field of centrifugal pump due mainly to its suitability for direct connection to elec. motor and to cheapness of combination. Large proportion of development of centrifugal pump has consisted in more accurate manufacture to designs perfected mainly as result of trial and error. Point of special importance is unwillingness of most engrs. to use motors which run at 3000 rpm. Main motive is safety. Yet impeller running at about 3000 rpm. will be less intricate and lighter than one running at half that speed and will, in general, be more efficient. Mixed-flow and axial-flow pumps have characteristics of importance in special applications: at const. speed, ratio of no-delivery head to normal greater than usual with centrifugal pumps, and power-absorption diminishes with decreasing head. Purely axial-flow pump somewhat less efficient than mixed-flow pump. Centrifugal borehole pump has advantage of reciprocating pump in steady rate of power absorption. For village supplies requirements are below best economic capac. of centrifugal borehole pump. Requirements may be efficiently met by automatically controlled bucket pump which may be entirely self-lubricating. Seems inevitable that use of direct current will decrease in future. Great disadvantage of ordinary induction motor is that it is not amendable to speed control. Where speed variation great, variable-speed drive will usually pay. Sturdy design of all devices used important, especially for automatic gears in unattended stations. Full diagrams should be available and all parts should be accessible. Elec. tariff being frequently higher during peak load periods, often desirable to limit pumping to off-peak times by use of clock-operated relay. Centrifugal pump and elec. motor reliable and replacement easy matter. Water engr. may be excused, who, in order to obtain immunity from strikes in other industries, insists on installing prime movers under his own control. Examples show economic advantage generally obtainable from unattended elec. station. *Discussion*. Wtr. & Wtr. Eng. (Br.) **49**: 85 (Feb. '46). *Author's addnl. remarks:* Necessary to get rid of (obsolete parts) of plant even though certain amount of residual value. Elec. plant can usually be housed easily in bldg. which formerly held any sort of plant, but often cheaper to knock down and build anew. PRESIDENT: Author's paper indicates that all needed in ideal distr. system is 1 or 2 small pipes and some scattered booster pumps. No doubt that capac. of main can be increased by putting booster pumps on it, but probably some distr. people will stand up for "white elephant" service reservoirs. Attractive theory to pepper site with boreholes and put pumps into those which prove satisfactory, if any do so. In filling storage reservoirs in such a manner as to avoid max. demand charges, sometimes necessary to take water when it comes down river and pump into reservoir. Variations in elec. charges over Metropolitan Water Board area extraordinary. Weakness of grid, as author pointed out, is that prime movers are not under control of water authorities. Oil engine stand-by desirable. Oil, however, is imported fuel; use should be discouraged as much as possible. L. F. HOBBS: Obviously wiser to take water at source for small community than to take it lower down from pold. river, purify it, and pump it back. Hezekiah, ancient water works engr., had no use for grid, and preferred gravitational system. Centrifugal pump and centrifugal well pump would have come along with or without grid. Versality of control gear and high eff. of well pumping plant of electrically driven type not advantages of grid itself. In striving for last ounce of efficiency in variable-speed motors, we are sometimes apt to overstep mark. Author was able to buy elec. at cheaper rates than some water works in home counties. Effect on author's conclusions is matter of detail and can be readily worked out. Author's figures on maint. cost of diesel plants at least 10% high. Author's price for oil was £11 per ton. In few months since paper written, price dropped almost 20%. E. S. BONIFACE: Engr. to find water must find fissures in chalk. Full development of good site in chalk can be done only by constr. of well and headings with subsidiary borings. B. L. MCMILLAN. How far would author push advocacy of multiple small pumping stations? Would multiple elec. pumping units show economy over conventional arrangement? N. J. PUGH. Au-

thor's advocacy of use of water mains for grounding local a.c. is impishness. To allow suggestion to pass unchallenged would be invitation to elec. industry to seek to justify practice which speaker considers grave menace to pipelines. HETHERINGTON. System of wells and headings cannot be entirely abandoned in favor of number of boreholes. P. G. H. BOSWELL. Compelled to take issue with author's suggestion that individual stations on chalk should be developed in form of series of boreholes rather than wells and adits. Always felt tinge of regret over failure to take suggestion to supply London with water by means of underground galleries aligned along strike of chalk and lower greensand. J. H. MAWSON. How to achieve safety? If, because grid makes it possible to use multitude of sources and then, for safety's sake, interconnect them; one is back to system of trunk mains and service reservoirs. Extensive system that can be put out of action at single stroke cannot be regarded as ideal. 2 methods of obtaining flexibility of output not touched on in paper: combination of 1 or 2 separate boosters with borehole pump, so that latter can be used alone or in series with either or both boosters; or, speed variation by means of frequency changing. *Author's reply:* Urged that, instead of laying expensive mains which will burst at comparatively low pressures, smaller mains should be judiciously used. No suggestion of abandonment of high-level reservoirs. Certainly more likelihood of contamn. of water in well into which man can enter than in borehole. In interest of freedom from contamn. as well as cheapness, several boreholes would be better than one well.—*H. E. Babbitt.*

Cleaning Reservoirs in Toronto. ANON. Wtr. & Sew. **84**: 1: 13 (Jan. '46). St. Clair Reservoir (50 mil.gal.), water el. 518 compared with L. Ontario el. 246, cleaned and inspected annually. Water delivered through 54″ steel pipe from high-level pumping station and circulation provided by specially designed reversed check valves on branches leading to 2 chambers of equal size normally operated in tandem. Max. water depth 23′ and floor slopes to valleys provided with sumps emptying into 20″ drain. Water for cleaning derived from 2.5″ line from 4″ main used for watering lawns and gardens overlying reservoir. Walls and columns washed thoroughly with fire hose and floor cleaned with squeegees. Concrete and joints inspected carefully and minor defects in latter calked with "Elastite." No defects of consequence found during 15 yrs. of operation. Before restoring to service, water admitted to depth of 2′, drained, filled and water tested for purity. Sterilization unnecessary. No permanent lights, illumination during cleaning being provided by floodlights on portable "cabtire" elec. cable. Cleaning and inspection normally requires 1 week. Rosehill Reservoir, uncovered and having 30-mil.gal. capac., cleaned thoroughly and repaired every 10 yrs.—*R. E. Thompson.*

Hot Water Distribution System for Reykjavik, Iceland. H. B. WHITE. J.Br.W.W.Assn. (Apr. '45). [Abstracted W.W.Eng., **98**:1254 (Oct. 31, '45)]. Term 'Company,' used in this article, means Municipality of Reykjavik, which is capital of Iceland. No private shareholders. Country very mountainous; still some active volcanos. In vicinity of these company has bored some 15 holes over fairly large area and tapped underground hot water which, upon forcing itself to surface, conveyed by 3″ pipes to small covered reservoir which has capac. of some 24,000 gal. One shaft inspected in which drilling still in progress. Drill then some 1200′ deep and water coming up from this shaft about 70°C. (158°F.). Estd. that at 1500′ water would be 90°C. (194°F.) and would be hot enough. In few places water had been found over bp. at 1000′. Adjoining the reservoir is recently built, clean and modern pumping station, which is some 10 mi. from Reykjavik. From here water pumped through two 14″ pipes to 4 concrete storage tanks, 24,000 gal. each, on outskirts of town. Pipes insulated by being 'lagged' and covered with pitch, and then filling concrete ducts with moss and soil. About every mile there are cross-connections, controlled by valves, which enable any section of main to be shut down, while still continuing supply to storage tanks through other main. About every 2 mi. expansion joints have been fixed. Three 300-hp. GE motors in pumping station, one for each main and one in reserve. Gages record quant. of water being pumped through each main, total quant. being some 5.4 mgd., pressure, which is between 150 and 175 psi., and temp. of water, which is around 90°C. (194°F.). Modern power station, with mostly American app., situated at one end of bldg. From storage tanks, two 17″ pipes deliver supply to town by means of gravity; avg. street main 3″ or 5″, and laid 3′ below ground in concrete ducts and insulated as before.

Usual size of supply pipe ¾″, although some business premises and hotels have up to 2″ supply pipe. Supply metered by use of American Hersey disc hot water meters inside premises, avg. temp. of water upon delivery being over 80°C. (176°F.) and pressure around 40 psi. Leaks in pipes soon noticed by steam that can be seen rising from ground. Street valves and stopcocks fitted as on cold water system. Charge to customer very cheap (considering high cost of living in Iceland), approx. 1s./240 gal. Also great saving in coal; company ests. that system has saved importation of 45,000 tons of coal per year. Since supply unlimited and maint. costs very low, company should derive great financial benefit for many years to come. About 3000 premises already connected and supply to remainder only governed by lack of material and labor. Company ests. that it supplies 30 gpd. per capita to pop. of Reykjavik and surrounding dist.—*P.H.E.A.*

Experience Regarding Wood-Stave Pipes. Stig Regnall. Svenska Vattenkraftföreningens Publikationer (Swedish), **385**:10 ('46). Although wood-stave pipe used in Sweden on comparatively large scale ever since end of last century, no summary previously made of experiences in field. In response to questionnaire circulated by Swedish Water Power Assn. in fall 1941, inf. given on operating experiences with about 200 wood-stave pipes used in Sweden. Questionnaire primarily concerned with pipe for hydroelectric stations, but data also obtained on small no. of pipes used in industrial water supplies. Inf. analyzed and results classified under 3 main headings of Design, Constr. and Operation; Decay; and Freezing. Before 1910, abutting end joints of staves connected by wood or metal collars and pipes banded with endless steel bands tightly stretched by wooden wedges. This constr. superseded by continuous stave pipes banded with round rods held together by band couplings, also called "shoes." During past few yrs., slightly modified continuous stave pipe, termed "group stave pipes," came into use in Sweden. Comprised of two groups of staves equal in length; every second stave belongs to same group, and is displaced by half length with relation to other group. Thus every second stave jointed in same cross-section of pipe. To reduce bearing pressure between bands and staves, use of flat steel bands resumed in recent yrs. Shoes now employed centrically made of pressed sheet steel or cast iron. Staves usually 60–80 mm. thick. About 90% of Swedish pipes carried on cradles or other supports; some, esp. those of large diam., rest on gravel or broken stone beds. Pipes for water power often 1670 m. long; max. length for industrial water supply 12,000 m. Largest diam. now in use, 3.5 m.; max. head, 59 m., but usually head lower, about 10 m. Life of pipe generally dependent upon damage caused by wood rot. Normal life of pipe correctly designed and properly maintained about 45 yr. in Sweden. Most pipes studied not treated with any timber preservative. Life in U.S. seems much shorter, probably due to climatic differences. Clay gives most favorable results of protective coverings employed. Risk of freezing less than with steel for equivalent conditions, and this danger decisive factor in detg. whether non-covered pipes can be used at all. Freezing all-important problem in use of wood-stave pipe in Sweden. Of 197 pipes studied, 53, or 27%, have frozen—13, or 7%, in service; 8, or 4%, completely clogged by ice. Theoretical investigation shows that $v_s = v \cdot \frac{D}{L}$, termed specific veloc. of water flow (where v is veloc. of water flow in pipe; D, diameter; and L, length of pipe), is factor of paramount importance in detg. whether pipe is liable to freezing. Higher the sp. veloc. of water flow, smaller the risk of freezing. Under normal intake conditions in Sweden, therefore, sp. veloc. of water flow in wood-stave pipes unprotected by covering should not be less than about 0.003 m./sec. Freezing takes considerable time, however, and ice can therefore be formed on inside of pipe without necessarily clogging whole pipe or seriously disturbing operation. Special attention directed to insulating effect of ice itself.—*Ed.*